

AN ANALYSIS OF THE IMPACT OF SEA
LEVEL RISE ON LAKE ELLESMERE – TE
WAIHORA AND THE L2 DRAINAGE
NETWORK, NEW ZEALAND

A thesis submitted in partial fulfilment of the requirements

for the Degree

Of Master of Engineering in Civil Engineering

in the

University of Canterbury

by

S. S. Samad

University of Canterbury

2007

Table of Contents

Acknowledgments.....	viii
Abstract.....	ix
1 Introduction	1
1.1 Objectives.....	4
1.2 Thesis Structure	5
1.3 Description of Methodology.....	5
2 Background	7
2.1 Overview of the L2 Drainage Scheme and Catchment	7
2.2 The Role and Description of Lake Ellesmere	10
2.3 Water Balance Analysis	11
2.4 The Role and Description of the L2 River	13
2.5 The Effects of Sea Level Rise.....	15
2.6 Flow Regime	16
2.7 Hydrological System	17
2.8 Surface Runoff.....	17
2.9 Hydrological Model (SCS Method)	18
2.10 Hydraulic Routing	19
2.11 Aquatic Vegetation and Resistance to flow.....	21
3 Fieldwork.....	24
3.1 Capacitance Water Level Probe Loggers	24
3.1.1 Site Selection.....	25
3.1.2 Calibration.....	28
3.1.3 Housing	30
3.1.4 Installation	32
3.2 Surveying	33
3.2.1 Site Selection.....	33
3.2.2 Surveying Methodology	34
3.3 Streamflow Gauging.....	38
3.3.1 Streamflow Velocity Methodology	38
3.4 Roughness Coefficient (Manning's n)	40
4 Analysis of Data	41

	ii
4.1 Lake Data	41
4.2 Hydraulic Gradient.....	42
4.3 L2 River Monitoring Data	44
4.3.1 Water Level Loggers	44
4.4 Surveying Results and Discussion.....	52
4.5 Streamflow Gauging Results and Discussion	56
4.6 Rainfall Data.....	62
5 Computational Modelling.....	64
5.1 Hydrological Model	64
5.1.1 Introduction.....	64
5.1.2 Meteorological Model.....	64
5.1.3 Basin Model	66
5.1.4 Control Specification.....	73
5.1.5 Calibration and Validation of Hydrologic Model.....	73
5.1.6 Sensitivity Analysis	79
5.1.7 Model Output	83
5.2 Hydraulic Model	83
5.2.1 Introduction.....	83
5.2.2 Model Setup.....	84
5.2.3 Network Module	84
5.2.4 Cross-Section Module	85
5.2.5 Boundary Module.....	86
5.2.6 Hydrodynamic Module	88
5.2.7 Model Settings.....	90
5.2.8 Model Output.....	90
5.2.9 Calibration and Validation of Hydraulic Model	90
5.2.10 Sensitivity Analysis.....	96
6 Discussion	102
6.1 Efficiency of the L2 Drainage System	102
6.2 Simulations of the Impact of Lake Level Rise coupled with large Rainstorm Events.....	111
6.2.1 Hydrologic Simulation of Various Scenarios	112
6.2.2 Hydraulic Simulation of Various Scenarios	118

7	Summary and Conclusions.....	126
7.1	Introduction	126
7.2	Objective and Methodology	126
7.3	Computational Modelling	127
7.4	Results of the Computational Modelling	127
7.5	Limitations in Measurements	128
7.6	Conclusions	128
8	Recommendations	131
	References.....	133
	Appendices.....	136
A.	Groundwater abstraction, surface water takes and discharge consents along the L2 River.....	136
B.	Low Flow Recommendation	137
C.	Loggers and Cross Sections Photographs	142
D.	Runoff Curve Numbers Table	149
E.	Sensitivity Analysis	152
F.	Photographs – Flood Events	155
G.	Drain Maintenance	165
H.	Resource Consent Conditions.....	169

List of Figures

Figure 1-1 Location of Lake Ellesmere, the L2 River and Banks Peninsula (Land Information New Zealand 1:50000 Topographic Map sheets).....	2
Figure 1-2 Representative cross-section of the lake, the sea interface and groundwater flow .	2
Figure 2-1 The L2 drainage catchment and scheme (Land Information New Zealand 1:50000 Topographic Map sheets).....	8
Figure 2-2 Land use within the L2 catchment (New Zealand land use cover database).....	9
Figure 2-3 Drains draining into the L2 River (supplied by SDC).....	14
Figure 2-4 Variables in the SCS Method of rainfall abstraction: I_a = initial abstraction, P_e = rainfall excess, F_a = continuing abstraction, and P = total rainfall (Mays 2005)	18
Figure 3-1 Odyssey capacitance water level logger	25
Figure 3-2 Photographs of the capacitance water logger monitoring sites	26
Figure 3-3 Site locations (1 through 4) (Land Information New Zealand 1:50000 Topographic Map sheets).....	27
Figure 3-4 Calibration of logger (Dataflow Systems Pty Ltd.)	28
Figure 3-5 Screenshot of Sensor Calibration	29
Figure 3-6 PVC shroud used to house the water level sensor	31
Figure 3-7 Waratah used for anchoring sensor in stream at site 1	32
Figure 3-8 Selected cross-sections along the L2 River (Land Information New Zealand 1:50000 Topographic Map sheets)	33
Figure 3-9 Cross-section showing surveying setup	35
Figure 3-10 Surveying cross sections along the L2 River using a kayak	35
Figure 3-11 TNL 4700/5800 (or R8) RTK system used to record elevation.....	36
Figure 3-12 Map showing location of where base stations were established.....	37
Figure 3-13 FP101 Global Flow Probe	39
Figure 3-14 Velocity Measurement	40
Figure 4-1 Kaitorete Barrier cross section showing relationship between lake levels and sea levels at lake opening cut to sea (Taylor 1996).....	42
Figure 4-2 Recorded water levels at sites 1, 2, 3 and 4 for the period of 25th July to 18th August, 2006	45
Figure 4-3 Minor variation in water level at site 4	46
Figure 4-4 Major variation in Site 4 water level.....	47
Figure 4-5 Correlation of the wind direction and water level for the period of 21 to 24 October 2006 (wind data provided by Environment Canterbury from their hydrotel site within Lake Ellesmere).	48
Figure 4-6 Wave moving upstream, due to wind (2006-12-7, 17:40 hrs)	48
Figure 4-7 Effect of south-easterly winds on the L2 River (Land Information New Zealand 1:50000 Topographic Map sheets)	49
Figure 4-8 Inundation of surrounding habitats	50
Figure 4-9 Weed stuck to the logger housing and support (2006-08-10), site 3	51
Figure 4-10 Deepest point of each cross section along the channel from site 1 to outlet (Lake Ellesmere), and the reaches along the river (between surveyed points).....	52
Figure 4-11 Slopes and observed water surface level for both winter and summer periods...	54
Figure 4-12 Cross-section profiles	55
Figure 4-13 Measured velocity profiles, site 1	56
Figure 4-14 Measured velocity profiles, site 2	57
Figure 4-15 Measured velocity profiles, site 3	57

Figure 4-16 Measured velocity profiles, site 4	58
Figure 4-17 Water Level Profile for the month of November	60
Figure 4-18 Relationship of discharge vs water level at different periods for site 1	61
Figure 4-19 The effect of Rainfall for the month of August	62
Figure 5-1 Three examples of the storm hyetographs used for calibrating the hydrological model	66
Figure 5-2 Physical representation of the L2 catchment showing subbasins, reaches and junctions.....	67
Figure 5-3 Measured vs. modelled storm event for the period of 13 to 16 August, 2006.....	76
Figure 5-4 Measured vs. modelled storm event for the period of 21 to 24 August, 2006.....	77
Figure 5-5 Measured vs. modelled storm events for the period of 03 to 07 October, 2006....	77
Figure 5-6 Potential flooding areas along the L2 River after an extreme rainstorm events....	78
Figure 5-7 Sensitivity analysis for peak flow, peak time and total volume for curve number81	
Figure 5-8 Sensitivity analysis for peak flow, peak time and total volume for initial abstraction.....	81
Figure 5-9 Sensitivity analysis for peak flow, peak time and total volume for lag time.....	82
Figure 5-10 Network editor showing the graphical view of the L2 River (Land Information New Zealand 1:50000 Topographic Map sheets)	85
Figure 5-11 Cross-section showing raw data and cross-sections of the L2 River	86
Figure 5-12 MIKE 11 time series editor showing graphical and tabular flow rate data for site 1	87
Figure 5-13 MIKE 11 boundary editor	88
Figure 5-14 Hydrodynamic parameters	89
Figure 5-15 Comparison of measured vs. modelled for the period of 25 to 31 July for sites 2 and 3.....	94
Figure 5-16 Comparison of measured vs. modelled for the period of 21 to 24 August using site 2 as an inflow parameter for sites 2 and 3	94
Figure 5-17 Comparison of measured vs. modelled for the period of 21-24 August using the HEC simulation as inflow parameter for sites 2 and 3.....	95
Figure 5-18 Comparison of measured vs. modelled for the period of 1-15 December for sites 2 and 3	95
Figure 5-19 Sensitivity analysis for water level for Manning's n	98
Figure 5-20 Change in elevation at cross-section 6.....	99
Figure 5-21 Samples of cross sections defined at 0.5, 1.0 and 2.5 m intervals used in the sensitivity analysis.....	100
Figure 5-22 Resulting water levels after defining all cross-sections at 0.5, 1.5 and 2.5 m intervals	100
Figure 6-1 Weeds within the channel looking downstream of Pannetts Road bridge.....	103
Figure 6-2 Weeds growing on the water surface, upstream of cross-section 9	103
Figure 6-3 Drop in water levels after partial weed clearing done in December	104
Figure 6-4 De-weeding of the L2 River between McDonald Road and Englishs Road	105
Figure 6-5 Cross-section (7) with water level close to overtopping the channel left bank. ..	107
Figure 6-6 Flooding in Yarrs Lagoon as a result of high water level associated with continued weed growth (January 2007).	107
Figure 6-7 Ellesmere Queen, the floating weed cutting plant (Lane 2004).....	109
Figure 6-8 Weeds within the drains at McDonald and Springs Road (January 2007)	110
Figure 6-9 Bank collapse due to the steep sides on bend within the L2 channel	111
Figure 6-10 IDF curve, natural log of Intensity as a function of natural log of Duration for the 2, 10 and 20 year ARI storm events	113

Figure 6-11 Redistributed precipitation using the second quartile method for the 2, 10 and 20 year ARI storm events	113
Figure 6-12 Rainfall intensity as a function of duration for the L2 catchment.....	114
Figure 6-13 Subbasins, junctions and outlet of the L2 catchment (Land Information New Zealand 1:50000 Topographic Map sheets)	115
Figure 6-14 Simulation of the 2 year ARI storm event during winter period.....	116
Figure 6-15 Simulation of the 10 year ARI storm event during winter period	117
Figure 6-16 Simulation of the 20 year ARI storm event during winter period	117
Figure 6-17 Summary of simulated results for the 2 year ARI storm event for winter and summer periods	119
Figure 6-18 Summary of simulated results for the 10 year ARI storm event for winter and summer periods	119
Figure 6-19 Summary of simulated results for the 20 year ARI storm event for winter and summer periods	120
Figure 6-20 Comparison of water levels (dashed red lines) for both winter (top) and summer (bottom) periods for the 10-year storm with a very high lake level	122
Figure 6-21 Flood depth (top) and the longitudinal view a 20-year ARI storm simulated during the winter period with a very high lake level.....	123

List of Tables

Table 2-1 Estimates of mean monthly water balance terms for Lake Ellesmere (Horrell 1992)	12
Table 3-1 Showing the various length of probe used and location	27
Table 4-1 Frequency and duration of lake openings at various opening levels 1945 – 2005	41
Table 4-2 Summary of opening statistics 1945 – 2005	41
Table 4-3 Projected Lake opening levels.....	43
Table 4-4 Water level limits for each sensor	45
Table 4-5 Comparison of water levels before and after the weeds have been removed	51
Table 4-6 Length and slope for each reach of the river surveyed	53
Table 4-7 Reduction of velocity coupled with a rise in water level due to weed growth	59
Table 4-8 Rating curves equation.....	61
Table 5-1 Initial abstraction used in the SCS curve number loss model	68
Table 5-2 Conversion of drain_class to hydr_soil	69
Table 5-3 Curve number used in the SCS curve number loss model	70
Table 5-4 Impervious used in the SCS curve number loss model	70
Table 5-5 Lag time for subbasins 1 to 4.....	71
Table 5-6 Average monthly baseflow contribution observed (m ³ /s) for July to December, 2006	72
Table 5-7 Reach Routing.....	73
Table 5-8 Calibration and Validation details for the HEC-HMS model.....	75
Table 5-9 Estimated Maximum flow (discharge) within each section using a range of velocity (0.30 to 0.50 m/s)	79
Table 5-10 The storm events used.....	80
Table 5-11 Calibrated Manning's n Values.....	89
Table 5-12 Calibration and validation.....	92
Table 5-13 Sensitivity Analysis Summary	97
Table 6-1 Summary table of hydrologic simulations for current conditions for both winter and summer period	115

Acknowledgments

I would like to express sincere appreciation to Dr Tom Cochrane and Dr David Painter, my supervisors for their advice, assistance and enthusiasm in the preparation of this manuscript. In addition, I would also like to acknowledge the following persons and organisations whose assistance has been invaluable in completing this thesis:

- Peter Coursey and Peter McGuigan for their assistance with the fieldwork in the cold and inhospitable conditions.
- Ian Sheppard and Kevin Wines for their technical support in the Fluid Mechanics Laboratory.
- Personnel from both Selwyn District Council and Environment Canterbury for providing available data and information.
- Selwyn District Council, New Zealand's International Aid & Development Agency (NZAID) and the University of Canterbury for their financial assistance.
- My wife for her moral support.

Abstract

The potential impact of sea level rise on Lake Ellesmere – Te Wiahora and the subsequent effect on the efficiency and performance of the L2 Drainage network was investigated in relation to the operation of the L2 Drainage scheme. Lake Ellesmere is currently manually opened for drainage to the sea when the lake levels reach 1.05 m above mean sea level (asl) in summer and 1.13 m asl in winter. With a rise in sea level, the lake opening levels for both summer and winter would have to increase in order to maintain the current hydraulic gradient. Higher lake levels would impact drainage schemes such as the L2 drainage network. An integral research approach was used to study this potential impact, including fieldwork, analysis of data, hydrologic and hydraulic modelling. Both the hydrologic and hydraulic response of the L2 catchment and river were reproduced with reasonable accuracy by the use of computational models. Simulations of 2, 10 and 20 year annual recurrence intervals (ARI) rainstorm events coupled with higher lake levels show increase flooding along the length of the river. An increase in the lake opening levels, coupled with southeasterly wind was shown to have increased the degree of flooding on adjacent farmlands, but only a 3.50 per cent increase of water level (for all conditions simulated) 3.5 km upstream of the L2 River. The study clearly shows that weed growth within the L2 River plays an important part in controlling the water level within the channel. Results show it was responsible for an observed water level rise of 0.30 m from the winter to summer season. The combined use of hydraulic and hydrological models provides an effective tool to study future impacts on the drainage efficiency and performance of the L2 drainage scheme and other similar systems. The potential for both models to be used as a predictive tool for improving the operation of the L2 scheme and Lake Ellesmere was only limited by the difficulty in estimating model parameters especially for the hydrologic model.

1 Introduction

“Lake Ellesmere (Te Waihora) is a large shallow, brackish coastal lake” (Taylor 1996) located south of Christchurch, adjacent to Banks Peninsula (Figure 1-1). The lake has been described as highly eutrophic. The consequences of eutrophication can include deoxygenation of the bottom waters and sediments, regular blooms of toxic algae, profuse weed growth and lack of salmonid fish (Wong 1999). It receives inflows from surface runoff via tributaries and drainage schemes, groundwater and seawater intrusions (Figure 1-2). The lake catchment extends from Banks Peninsula to the foothills of the Southern Alps, roughly between the Waimakiriri and Rakaia Rivers, and covers an area of 276000 ha. Seventy five per cent of the catchment is flat to undulating and twenty per cent of this land has been described as highly productive (Taylor 1996). There are around 40 minor tributary streams and drains discharging directly into Lake Ellesmere. These include streams such as Harts Creek, Irwell River, Kaituna River, Prices Valley Stream, Doyleston Drain and the L2, which is of particular interest to this research.

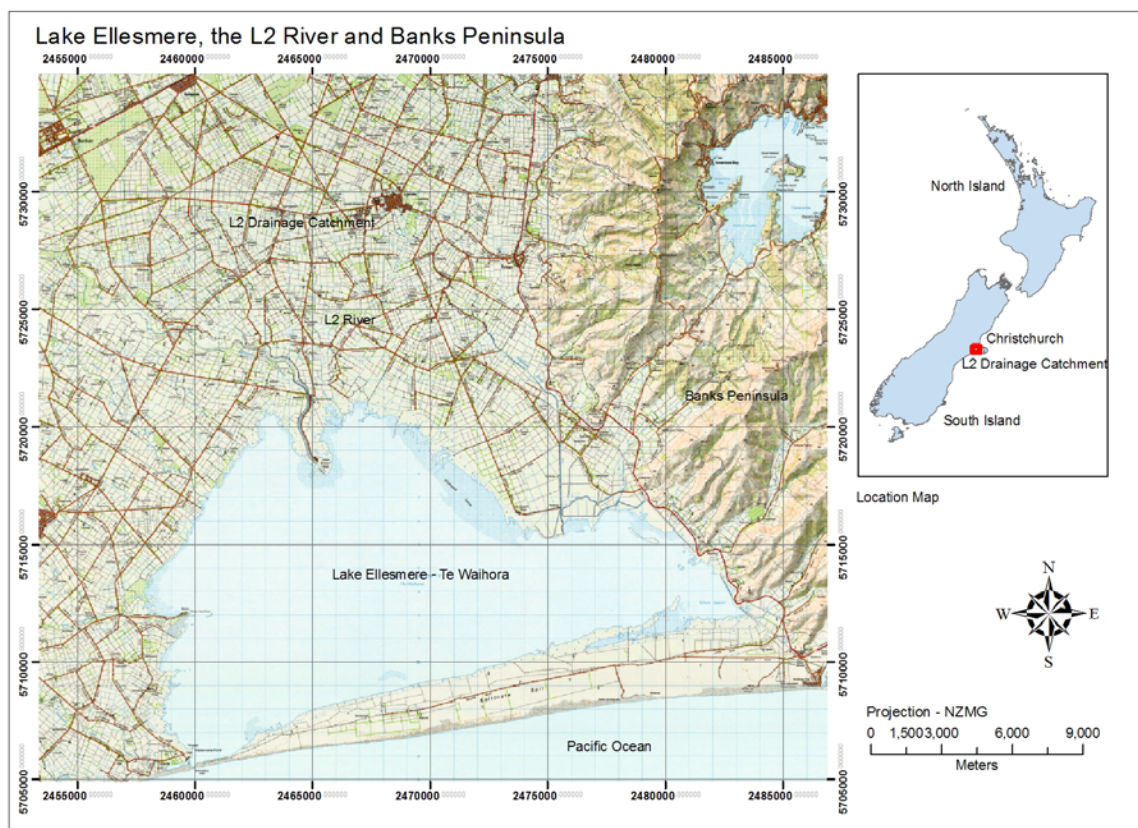


Figure 1-1 Location of Lake Ellesmere, the L2 River and Banks Peninsula (Land Information New Zealand 1:50000 Topographic Map sheets)

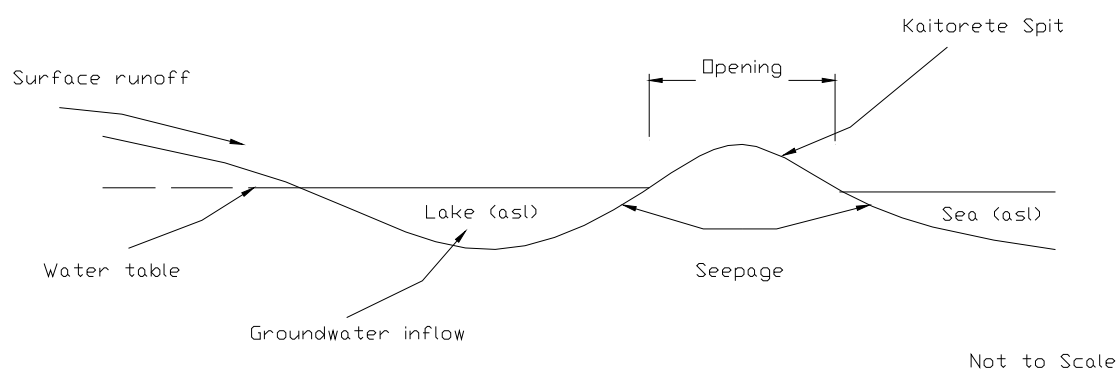


Figure 1-2 Representative cross-section of the lake, the sea interface and groundwater flow

The L2 drainage catchment is located within the Selwyn District Council (SDC) and is predominantly a highly productive farming area. The catchment has an area of 5067 ha between Rattletrack and Powells Roads and has a total length of classified drains of 64.4 km. The main drain for this scheme is the L2 River that flows from Lincoln township to Te Waihora; this is approximately 10.3 km (Iremonger 2005).

As the lake level rises, the surrounding land floods and the drainage system in the catchment responds more slowly. In order to prevent flooding and improve drainage in the catchment, Lake Ellesmere is opened on average three times per year to the sea by using heavy machinery to create a passageway between the lake and the sea (Kaitorete Spit) (Figure 1-2). This manual method of opening the lake to the sea has been used since the 1800s and is the most cost-effective way to reduce the level of the lake.

A higher mean lake level would have more advantages than disadvantages for the lake ecosystem. However, the higher lake level would negatively affect agricultural use of the adjacent farmlands (lake margins).

Climate change, however, could have an impact on the level of the lake or on the number of times the lake has to be opened. A study in the *US journal Science* suggests a rise in sea level of several metres could be reached before the end of the century (Ricon 2006). This study combines computer models of rising temperatures with records of the ancient climate that indicate that average sea levels around the world could rise by up to 6 m (20ft) by 2100, placing millions of people at risk. Mean sea levels around New Zealand have already shown a 10 cm to 20 cm rise over the past 100 years and it is expected to rise between 9 to 88 cm¹ by the end of the century (Ministry for the Environment). The rise in sea level will have an

¹ Extreme values from Ministry for the Environment report

impact on Lake Ellesmere. This in turn will have an effect on the efficiency and performance of the drainage network.

In order to study the potential impact of lake level rise on the efficiency and performance of the drainage networks within the region, this research has focussed on the L2 catchment which is representative of the drainage networks found within the Lake Ellesmere catchment. The reasons for studying the L2 district are the simplicity of the drainage network, it is representative of other drainage networks within the Lake Ellesmere catchment, availability of data, resources and the size of the district. The L2 is also the largest single source of fresh water flowing into Te Waihora/Lake Ellesmere (Maw 2004).

The research was carried out by field work and hydrologic (HEC-HMS) and hydraulic (MIKE 11) modelling. This project was carried out in collaboration with the Selwyn District Council (SDC), the Waihora Ellesmere Trust and Environment Canterbury (ECan) and serves as a case study to understand similar drainage problems around Lake Ellesmere.

1.1 Objectives

The general objective for this thesis is to understand the flow dynamics of the drainage network so that the behaviour of the L2 drainage system can be predicted as Lake Ellesmere levels fluctuate.

The specific objectives are:

1. To investigate the efficiency of the existing L2 drainage system.
2. To simulate drainage conditions when the lake level rises.

3. To analyze and evaluate the water balance between the sea, lake and the L2 drainage system.
4. To recommend guidelines for improving the drainage of the L2 District.

1.2 Thesis Structure

Background information on Lake Ellesmere and the L2 River is given, followed by a review of relevant literature.

Descriptions of methodology and results are presented in separate sections, for each of the three phases of the project: fieldwork, analysis of data and computational modelling.

The results of all three phases are then integrated and discussed in the context of the thesis objectives.

1.3 Description of Methodology

An integrated combination of fieldwork, data analysis and computational modelling was used in this study to investigate the impact of sea level rise on Lake Ellesmere – Te Waihora and the L2 drainage network.

Capacitance water level loggers were installed; water level data were obtained for the purposes of calibration of the models, the construction of flow-height (Q-h) rating curves and the calculation of lag times. Cross-sections of the river were selected and surveyed to gather the information required for the building of the hydraulic model. Streamflow gauging was also carried out to aid in the construction of the Q-h rating curves. The time spent working on the river provided insights into the extent and volume of storage during periods of high flow.

Analysis of data obtained from the capacitance water level, lake opening, rainfall, and streamflow was undertaken. Results from this phase of the analysis were used as the verification data for the computational modelling, and raw data were used to set boundary conditions for the models.

Computational modelling allowed for the effects of varying boundary and initial conditions to be investigated. A properly constructed and calibrated computational hydraulic model is equivalent to having a water level logger and a rating curve at every cross-section (Dark 2005).

2 Background

2.1 Overview of the L2 Drainage Scheme and Catchment

The L2 catchment, which was defined by the drainage network, is located on the east side of the South Island, south-west of Christchurch and is delineated in Figure 2-1. It has a total area of approximately 8271 ha (20438 acre) and is part of the larger Lake Ellesmere catchment where the L2 River drains into the Lake.

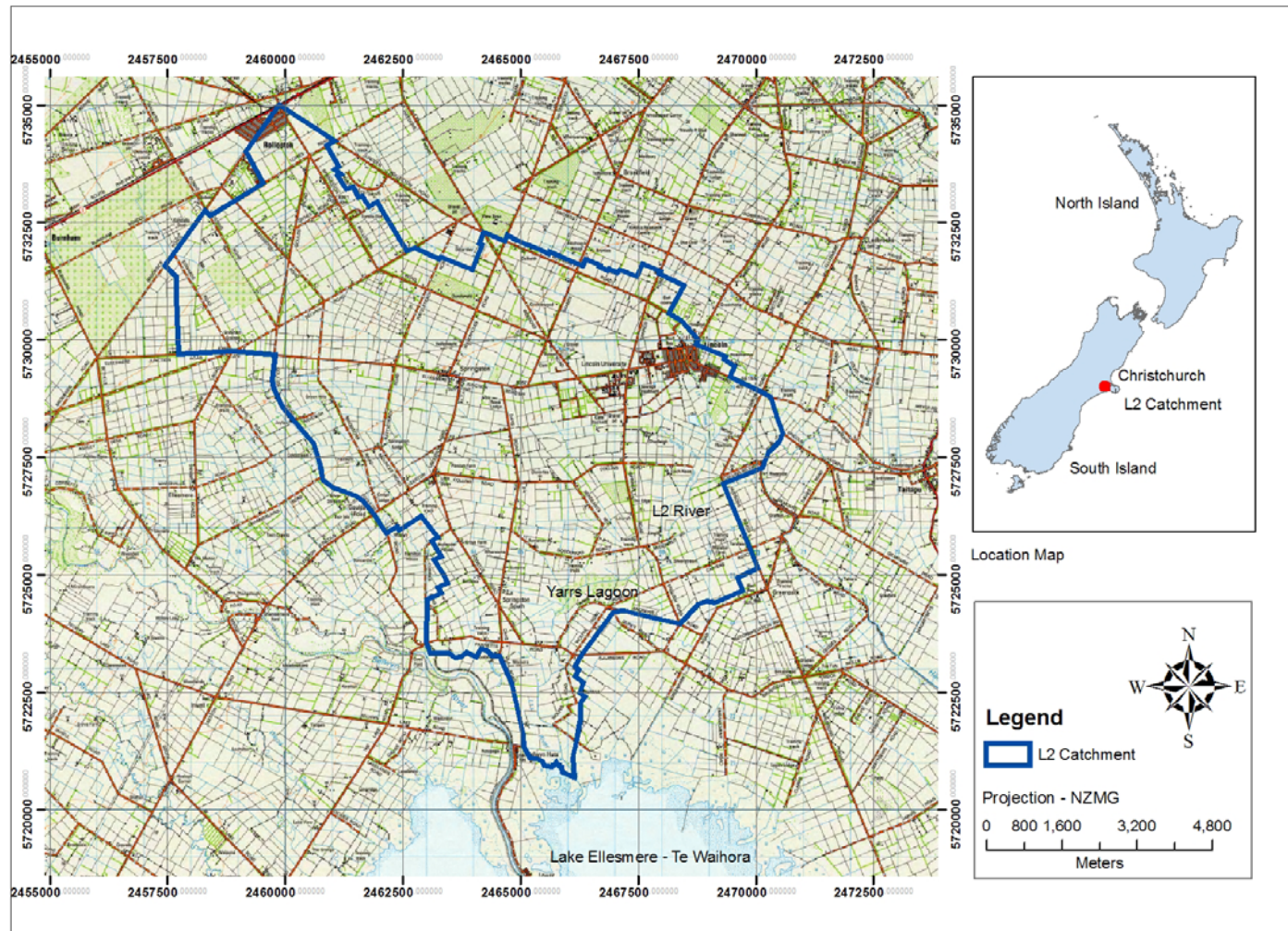


Figure 2-1 The L2 drainage catchment and scheme (Land Information New Zealand 1:50000 Topographic Map sheets)

Pasture for grazing is the predominant land use within the catchment. Horticulture is the second largest land use, dairying is a minor use of land within the catchment and lifestyle blocks make up the remainder (Figure 2-2). The study area is very gently rolling land. The L2 River rises from a significant spring east of the Lincoln township. The stream forms part of an intensive drainage network (L2) and is also used as a source of irrigation and stock water. There are a large number of surface water takes, discharge consents and groundwater abstraction wells in the catchment, which are also used for irrigation purposes (Appendix A).

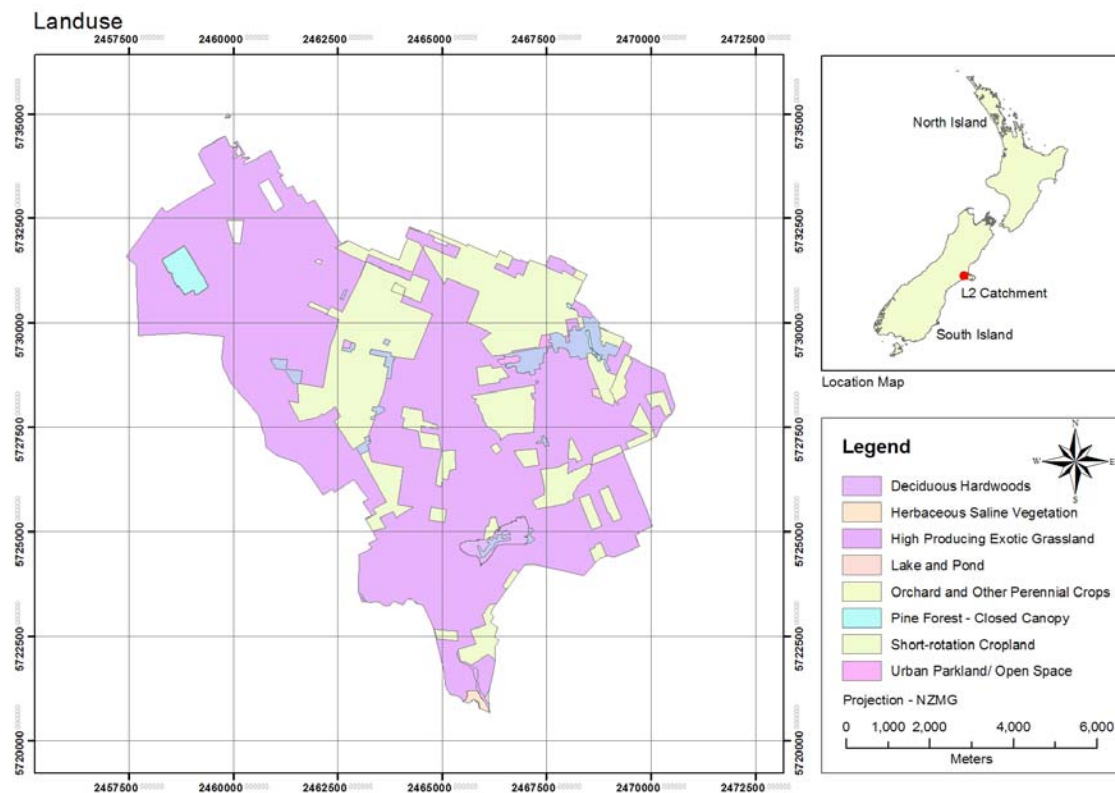


Figure 2-2 Land use within the L2 catchment (New Zealand land use cover database)

The direction of flow of groundwater in the L2 catchment is to the east, south-easterly direction towards the coastline. Groundwater originating from the Waimakarri River either flows below the confining strata into the aquifers, or above the strata into near-surface gravel

channels at the unconfined-confined aquifer boundary. These channels contain springs that form the source of the L2 River (Rosen and White 2001). The depth to groundwater varies throughout the catchment from between 0.2 m and 4 m in depth with a seasonal variation accounting for minor fluctuations (Rosen and White 2001).

The local authority that has jurisdiction and manages the L2 drainage scheme is the Selwyn District Council (SDC). It has overall responsibility for the activities and developments in the area. The Selwyn District is part of the region covered by Canterbury Regional Council.

2.2 The Role and Description of Lake Ellesmere

Lake Ellesmere is one of New Zealand's most important wetland systems, particularly in regard to wildlife habitat, and was recognized in 1981 by the International Union for the Conservation of Nature as being of international importance (Taylor 1996). The bed of the lake was returned to Ngai Tahu by the Crown as part of the Ngai Tahu Claims Settlement Act 1998. These water bodies are of significance to the local runanga for spiritual and food gathering reasons. The local and regional communities attach values to these waterbodies and anglers and other recreational users also use them (Shaw 2006).

Te Waihora (meaning 'water spread out') is regularly opened to the sea artificially to prevent flooding of adjacent land which is used for farming. At least 20 per cent of the land area is considered to be highly productive (Taylor 1996). In order to prevent flooding and improve drainage in the catchment, Lake Ellesmere, (the fourth largest lake (in surface area) in New Zealand) is opened on average three² times per year to the sea by the use of heavy machinery to reduce the level of the lake.

² For the period 1945 – 2005.

The lake is very shallow, with the natural deepest parts being just over 2.7 m at the eastern end of the lake and the average depth is 1.4 m. The large surface area can be attributed to the shallow grade of the land. At a level of 0.65 m above mean sea level (asl) the lake occupies an approximate area of 18000 ha while at 1.05 m asl the lake occupies an approximate area of 20685ha (Taylor 1996). There is no permanent outlet for Lake Ellesmere. It has been estimated that without human intervention the lake level would rise as high as 4 m asl, approximate area of 38900 ha, before naturally breaching the Kaitorete Spit to provide an outlet to the sea (Horrell 1992).

In 1947 the North Canterbury Catchment Board became responsible for lake management and instituted a management regime that required the lake to be opened when its height above mean sea level reached 1.05 m in the summer and 1.13 m in the winter (Taylor 1996). The variation in the level of Lake Ellesmere has been a source of concern to adjacent farmers since settlement, especially during strong south-westerlies which can raise the lake level on the leeward shore by more than 0.6 m (Taylor 1996).

2.3 Water Balance Analysis

Horrell (1992) has shown that the level of Lake Ellesmere is a complex balance between inflows (from streams, sea incursions, groundwater seepage, rainfall and seepage through Kaitorete Spit) and outflows (evaporation, artificial breaching of Kaitorete Spit, as well as seepage through Kaitorete Spit). Horrell gives the water balance of Lake Ellesmere as:

$$(I_t + I_r + I_g + I_{as} + I_{rs}) - (O_s + O_e + O_a) = \Delta S \quad 2-1$$

where:

I_t = inflow of surface water from lake tributaries

I_r = rainfall over the lake surface

I_g = groundwater inflow through the lake bed and walls

I_{as} = sea incursions resulting from artificial openings

I_{rs} = sea incursions during rough weather

O_e = evaporation from the lake surface

O_a = outflow from artificial breaching of Kaitorete Spit

O_s = outflow by seepage through Kaitorete Spit, and

ΔS = change in lake storage

While seepage through Kaitorete Spit occurs in both directions, according to the relative differences in lake and sea level, the net flux is as outflow (O_s). Because of the marked seasonality evident in a number of parameters in Equation 2-1, Horrell (1992) analysed the lake water balance on a monthly basis (Table 2-1).

Table 2-1 Estimates of mean monthly water balance terms for Lake Ellesmere (Horrell 1992)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
I_t	6.86	6.41	8.98	9.28	11.67	11.61	15.57	21.35	16.32	18.41	11.32	9.43
I_r	3.37	3.31	3.55	3.81	3.64	3.85	4.58	4.18	2.50	3.07	3.87	3.29
I_g	0.01	0.02	0.06	0.08	0.11	0.12	0.12	0.12	0.11	0.09	0.07	0.06
I_{as}	0.80	0.35	0.81	1.56	2.14	3.37	4.25	4.33	3.52	2.35	2.91	1.84
I_{rs}	0.78	0.76	0.96	1.44	1.81	2.09	1.64	1.69	1.14	1.04	0.98	0.76
O_s	0.69	0.69	0.87	1.09	1.32	1.57	1.59	1.01	0.96	0.86	0.75	0.70
O_e	12.94	10.65	7.11	4.02	2.04	1.53	1.66	2.04	4.20	7.19	8.94	10.61
O_a	1.51	1.58	3.75	5.58	14.25	13.54	30.88	26.58	18.29	18.52	7.99	12.24
L_h	0.705	0.705	0.754	0.815	0.878	0.945	0.951	0.792	0.78	0.753	0.722	0.708

Note: Storage term (ΔS) given as lake height, L_h (m asl). See Equation (2-1) for the description of other notation. All values in m^3/s .

As shown in Table 2-1, the winter period shows the largest inflow of surface water from lake tributaries into Lake Ellesmere as well as the largest amount of rainfall over the lake, with

the month of August yielding the highest. The period from January to April contributed the least amount of inflow into Lake Ellesmere. Inflow of surface water from lake tributaries, for example the L2 River, contributes most water to the lake compared to groundwater inflow through the lake bed and walls.

2.4 The Role and Description of the L2 River

The L1 rises on the northern side of the Lincoln township, whilst the L2 rises from a significant spring east of the Lincoln township. Ultimately, the L1 tributary drains into the L2, which makes its way to Te Waihora/Lake Ellesmere. The L1 'Liffey Stream' converges with the L2 River approximately 1 km south of the Lincoln township. Since they are both from springs in or around the Lincoln township area, surface flow is therefore a mixture of groundwater-sourced flow, and surface flow. Discharges from the Lincoln urban area would therefore expect to be contributing significantly to the flows of the L1 and L2 during and immediately after a storm event.

The L2 therefore, provides drainage for the L2 subdivision and the Lincoln urban area, as well as providing amenity, recreational and cultural value. Based on the results of an angling survey (NIWA 2003), the L2 is the second most popular fishing river in the Ellesmere/Selwyn catchment (with the Selwyn River being the most popular). The L2 is considered to be a good habitat for trout (as indicated in Fish and Game evidence).

Cattle and sheep access the river for water because many of the paddocks adjoining the waterway do not have a reticulated supply and are not fenced. The L2 is the largest single source of fresh water flowing into Te Waihora/Lake Ellesmere (Maw 2004).

It is a significant contributor to Lake Ellesmere, with the average flow being 2 cubic meters per second. The nature of the L2 River is such that there are no structures or crossings over

or in the river except for the four main road bridges crossing the river. Two of these are within the Lincoln township. There are many drains entering the L2 River as shown in Figure 2-3.

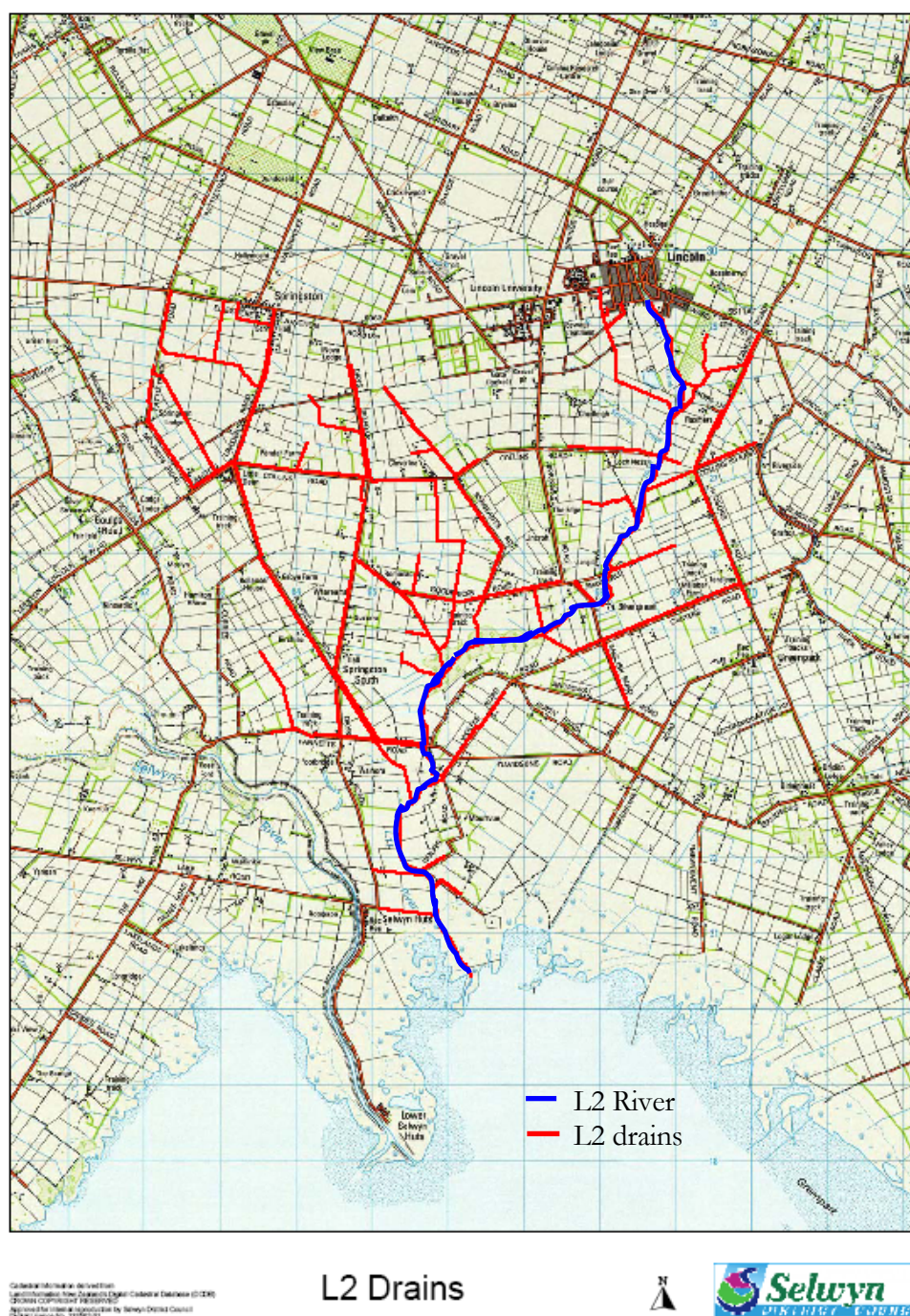


Figure 2-3 Drains draining into the L2 River (supplied by SDC)

The river system is highly modified, which in turn affects the ecology. The effects of this modification have been exacerbated by the frequent channel cleaning using hydraulic diggers. The frequent cleaning has resulted in a deepened channel with steep sides and in the physical removal of stream life.

In a natural stream there is a sequence of pools and riffles, and the channel follows in a sinuous path, rather than being straight as observed in the L2 River. A sinuous path provides overhanging banks, varying water depths, varying water velocity and a wide range of habitat for all the potential species to find a niche in. In some reaches of the L2 River willows dominate, while *Carex secta* and *Juncus spp* form riparian communities in some areas that are not adversely affected by grazing and cultivation.

2.5 The Effects of Sea Level Rise

Our climate is changing and in ways which affect all of us, with warmer temperatures, rising sea levels, more floods and droughts, and stronger winds (Ministry for the Environment 2006b). Many scientists believe that because of emissions already in the atmosphere, sea level rise is inevitable, irrespective of future greenhouse gas emissions. Oceans will continue to heat up causing the melting of ice, thus resulting in the rise of sea levels, even if the atmosphere itself is no longer becoming any warmer. Because of this, we cannot predict how much the climate will change in the future. We are sure that sea levels will rise but the quantity of this rise is unknown.

A rise in sea levels and greater frequency of severe storms may cause greater coastal erosion, inundation and saltwater intrusion into freshwater aquifers. Mean sea levels around New Zealand have already shown a 10 cm to 20 cm rise over the past 100 years and it is

expected to rise by 9 to 88 cm³ by the end of the century (Ministry for the Environment).

The Mean High Water Spring (MHWS) level (currently 0.92 m asl), around Lake Ellesmere will directly increase as a result of this rise in sea level. Because of this anticipated rise in the MHWS, the hydraulic gradient which produces an outflow of Lake Ellesmere to the sea at Kaitorete Spit (where the lake is opened to the sea) during high tides will be reduced.

There are many proposals for the raising of the current opening targets for ecological reasons. A higher mean lake level would have more advantages than disadvantages for the lake ecosystem. The lake level affects the composition, abundance and distribution of flora and fauna of Lake Ellesmere. A higher opening lake level would increase nutrient concentrations in the lake, because the frequency of lake openings would decline and thereby reduce the degree of flushing of the lake.

2.6 Flow Regime

The L2 River is classified as a perennial river and has one distinct flow regime. It has an average flow of 2 m³/s. The major inflows are from the drainage ditches along the river as well as from the springs present in or around the Lincoln township area. Lowflow assessment was undertaken by Environment Canterbury (ECan) in 2000 and 2004 for the L2 (Refer to Appendix B). Minimum flow recommendations were made in the order of 400 l/s for the site at Moirs Lane, and 1450 l/s at Wolfes Road, following the survey in 2000. The minimum flow at both sites was lowered in 2004 to 200 l/s and 1000 at each site respectively (the latter at Pannetts Road, just upstream of Wolfes Road).

³ Extreme values from Ministry for the Environment report

2.7 Hydrological System

Chow, Maidment, and Mays (1988) defined a hydrologic system as a structure or volume in space, surrounded by a boundary that accepts water and other inputs, operates on them internally, and produces them as outputs. The structure or volume is the totality of the flow paths through which water may pass as throughput from the point it enters the system to the point it leaves. For the L2, the main hydrological inputs for the catchment are rainfall, distributed in time and space over the catchment, runoff into the drainage network, then into the main L2 River and output into Lake Ellesmere.

Rainstorms can vary significantly in space and time. Rainfall hyetographs are plots of rainfall depth or intensity as a function of time. The spatial and temporal variations of rainfall and the concurrent variation of the abstraction process define the runoff characteristics from a given storm (Mays 2005). The difference between the total rainfall and the rainfall excess is the abstractions or losses, therefore losses are primarily water absorbed by infiltration with some allowance for interception and surface storage. After the local abstractions for an area of the catchment, water begins to flow overland as an “overland flow” and eventually into a drainage channel. At this point the hydraulics of the drainage network has a large influence on the runoff flow characteristics.

2.8 Surface Runoff

Overland flow occurs when the rainfall rate exceeds the abstraction and sufficient water ponds on the surface to overcome surface tension effects and fill small depressions (Mays 2005). Overland flow is surface runoff that occurs in the form of sheet flow on the land surface without concentrating in clearly defined channels (Ponce 1989).

2.9 Hydrological Model (SCS Method)

The US Department of Agriculture Soil Conservation Service (SCS) (1972) developed a rainfall-runoff relation for catchments and is the most used and proven method used in calculating runoff from a catchment. For any given storm, the depth of excess precipitation (P_e) or direct runoff, is always less than or equal to the depth of precipitation P . Hence after runoff begins, the additional depth of water retained in the catchment F_a , is less than or equal to some potential maximum retention S (Mays 2005) (Figure 2-4). For initial abstraction (I_a) before ponding, there is some amount of rainfall for which no runoff will occur. The SCS method assumes that the ratio of actual losses (F_a) to potential losses (S) is equal to the ratio of direct runoff (P_e) to potential runoff ($P - I_a$).

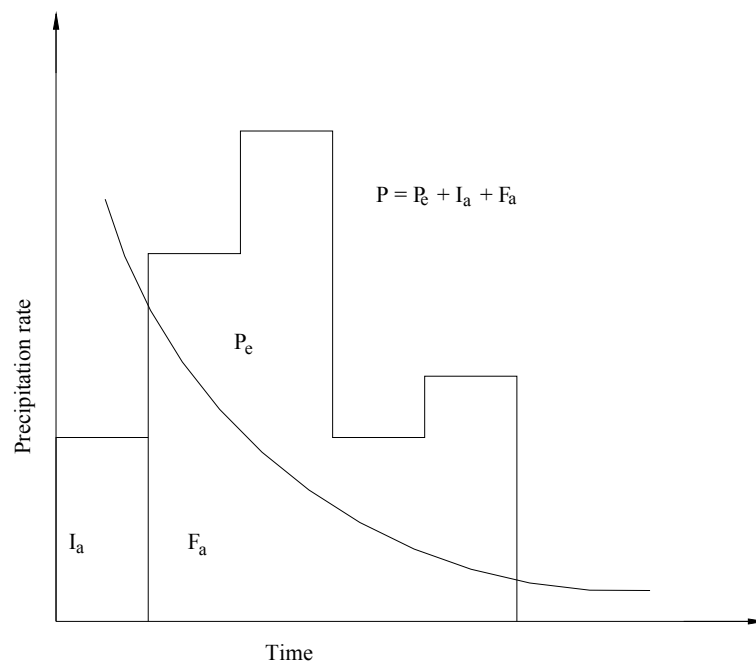


Figure 2-4 Variables in the SCS Method of rainfall abstraction: I_a = initial abstraction, P_e = rainfall excess, F_a = continuing abstraction, and P = total rainfall (Mays 2005)

$$P_e = \frac{(P - I_a)^2}{(P - I_a) + S} \text{ (from the assumption and continuity)}$$

$$I_a = 0.2S \text{ (empirical studies)}$$

$$P_e = \frac{(P - 0.2S)^2}{P + 0.8S} \text{ (from the two equations above)} \quad 2-2$$

$$S = \frac{1000}{CN} - 10 \text{ (graphical approximation to empirical data)}$$

(This is only true for S in inches)

The SCS method uses the runoff curve number CN, which is a function of cover type (land use), hydrologic condition, hydrologic soil group (A: deep sand, deep loess, aggregated silts, B: shallow loess, sandy loam, C: clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay, and D: soils that swell significantly when wet, heavy plastic clays, and certain saline soils) and antecedent soil moisture to determine S. The curve number is a dimensionless number defined such that $0 \leq CN \leq 100$. For impervious and water surfaces (all runoff) $CN = 100$ and for no runoff $CN = 0$.

2.10 Hydraulic Routing

The process of using a known upstream hydrograph(s) to determine the time and magnitude of flow at a specified downstream location is known generally as flow routing (Mays 2005). Routing by lumped system methods is called hydrologic or lumped routing; outflow as a function of time at a single location. Whereas routing by the distributing systems method is called hydraulic or distributed routing; flow rate and water surface elevation (depth) are calculated as a function of time and space throughout the modelled system which is also known as unsteady flow routing (Mays 2005); (Chow et al. 1988).

Hydraulic routing is based upon the one-dimensional unsteady flow equations referred to as the Saint-Venant equations, which are partial differential equations for continuity and momentum:

$$\frac{\partial(Uy)}{\partial x} + \frac{\partial y}{\partial t} = 0 \quad 2-3$$

$$g \frac{\partial y}{\partial x} + U \frac{\partial U}{\partial x} + \frac{\partial U}{\partial t} - g(S_0 - S_f) = 0$$

<i>Pressure</i>	<i>Convective</i>	<i>Local</i>	<i>Gravity</i>	<i>Friction</i>	2-4
<i>force</i>	<i>acceleration</i>	<i>acceleration</i>	<i>force</i>	<i>force</i>	
<i>term</i>	<i>term</i>	<i>term</i>	<i>term</i>	<i>term</i>	

In equations (2-3) and (2-4), U is the one-dimensional flux velocity, S_0 is the bed slope, S_f is the friction slope, x is the distance downstream and y is the depth perpendicular to the bed. The equations are derived assuming that the velocity profiles in the flow are one-dimensional and that the pressure distributions are hydrostatic along the lines normal to the channel bottom (Hunt 1995).

Equation (2-4), also known as the dynamic wave equation, is used when the inertia of the water body over time and space is important as is the case in river systems where the water surface slope, the bed slope and the bed resistance forces are small. The hydraulic features of the river reach represented by the computational point are generally lumped into the resistance term of the momentum equation (Equation 2-4). This resistance term is of utmost importance because it is based on an empirical law and as such is the only truly “adjustable” term during calibration (Cunge et al. 1980).

2.11 Aquatic Vegetation and Resistance to flow

The vegetation along the bed and banks of rivers plays an important role in the hydrodynamic behaviour, on the ecological equilibrium and on the characteristics of the river (Righetti and Armanini 2002). The vegetation can be classified into two different categories as far as flow resistance is concerned: plants having a height lower than the flow depth, and plants, the height of which is the same as the flow depth or higher.

The resistance due to vegetation can be described as a wall shear stress and the related roughness coefficient can be expressed as a function of vegetation height and of some biomechanical vegetation characteristics (Kouwen et al. 1969); (Kouwen 1988). For plants whose height is equal to the magnitude of the flow depth or higher, for example bushes or trees, the equivalent resistance can be evaluated as the combined effect of the hydrodynamic drag of the single plants (Petryk and Bosmanjian 1975).

The laws of flow resistance in open channels are essentially the same as those in closed pipes (Henderson 1966). In closed pipes, the flow is driven by a pressure gradient along the pipe, whereas in open channel flows, fluid is propelled by the weight of the flowing water resolved down a slope (Chanson 1999). Studies have suggested that the spatial location of plants within a channel affects the flow resistance, resulting in the claim that macrophytes⁴ contribute to local flooding.

Aquatic and riparian vegetation influence stream and river ecosystems – the amount of and composition on vegetation within a river or stream and their associated riparian corridors can substantially affect water velocities (both flood flow and baseflow), habitats, nutrient,

⁴ Macrophytes may exhibit a submerged or non-submerged canopy (Stephan and Gutknecht, 2002). They have roots and differentiated tissues and may be emergent (cattails, bulrushes, reeds, wild rice), submergent (water milfoil, bladderwort) or floating plants (duckweed, lily pads).

dynamics, water clarity and quantity, and ultimately the abundance and diversity of other aquatic life (Reeves, et al., 2004).

Several algebraic expressions have been developed with the aim of plotting the velocity profile above aquatic vegetation. The velocity distribution within the vegetation mainly depends on the types of vegetation (natural vegetation, artificial roughness elements) and on the density and arrangement of the plants (Stephan and Gutknecht 2002).

The first successful empirical formulae to calculate resistance were derived in the 18th century (Chezy formula) and the 19th century (Gauckler-Manning formula) (Chanson 1999). Other methods include the Strickler coefficient, Manning's roughness coefficient and the Darcy friction factor (more used in pipe flows). Manning's n is the most commonly used of the abovementioned roughness coefficients, and will be utilised in this research.

Manning's n can vary considerably with discharge at a particular site. Values of roughness coefficient can be estimated by a "visual comparison" approach, employed in their own right in hydraulic calculations, or as base values. Estimates of the roughness coefficient can also be obtained by using empirically derived predictive equations (Hicks and Mason 1991).

The value of Manning's n for a section of a stream is calculated using Equation 2-5.

$$n = \frac{KR^{2/3}S_e^{1/2}}{V} \quad 2-5$$

where,

$$S_e = \frac{\Delta h}{l} + \frac{\Delta h_v}{L}$$

$$h_v = \frac{\alpha V^2}{2g}$$

where, K is a factor to keep the equation dimensionally correct ($1\text{m}^{1/3}\text{s}^{-1}$), R is the hydraulic radius (m), S_e is the slope of the energy grade line (dimensionless), Δh is the difference in water surface elevation (m), V is the mean channel velocity (m/s), l is the length of river channel reach (m), L is the distance between the first and last cross-sections (m), Δh_v is the upstream velocity head minus downstream velocity head (m) and h_v is the velocity head (m).

Sediment can be deposited in areas where the flow has been decreased through excessive macrophyte growth. When sediment is deposited in these areas it provides a ready nutrient source and rooting medium for the macrophytes, this in turn causes further growth of the macrophytes and further reduces the hydraulic efficiency and increases sediment deposition (Taylor 2005). Morris and Dunderale (1996) found that discharge increased from 0 per cent to 38 per cent with 40-80 per cent weed removal (Morris and Dunderdale 1996). Experiments in Birdlings Brook, Canterbury, showed water level reductions of 0.5-1 m after partial and full clearance of weeds in the stream which can have a bankfull depth of 1.5 m (Hudson 2005).

3 Fieldwork

Extensive fieldwork was necessary to measure the L2 River bed slope, cross-sectional geometries, channel roughness coefficient (Manning's n), flow velocities and water levels to enable calibration and modelling with the computational (hydrologic and hydraulic) modelling.

3.1 Capacitance Water Level Probe Loggers

A capacitance water level probe logger is a device that records the water level at set time intervals when installed in rivers or streams. These devices provide temporal water level data necessary for the calibration of the computational models, for understanding the flow dynamics and to calculate the flow rate when velocity measurements were also taken. Four Odyssey capacitance water level loggers manufactured by Dataflow Systems Pty Limited were installed at selected locations along the L2 River to obtain water level data. The water level sensors are available in varying lengths and have an estimated precision of ± 0.005 m.

The capacitor consists of two conducting plates or cylinders separated by Teflon, (a dielectric insulating material which has good long term stability). The value of the capacitance (when the distance between the plates is fixed) is directly proportional to the area of the two plates in the capacitor. The stability of the dielectric material governs the stability or quality of the capacitor. Teflon has zero moisture absorption; its characteristics are therefore not altered by water immersion (Dataflow Systems Pty Ltd.). The Teflon-covered measuring element forms one plate of the capacitor while the second plate is the water in which the probe is immersed. As the water level varies, the area of water that is in contact with the Teflon surface also varies. The water is like a cylinder that is moving up and down the cylindrical Teflon-lined element. Hence the variation in capacitance is directly

proportional to the height variation of the water in contact with the Teflon. The brass counterweight at the base of the sensor element is also used to make electrical contact with the water (Figure 3-1).



Figure 3-1 Odyssey capacitance water level logger

The capacitance value is measured by the electronic module that is mounted at the top of the probe and recorded by the recorder that is also included in the electronic module. This module converts the value of the capacitance into a digital signal so that it is measured by the data recorder. Each time the recorder reads the sensor, a value is stored in the log.

3.1.1 Site Selection

Four sites were selected along the L2 River for the installation of the capacitance water level probe loggers (Figure 3-2). These monitoring sites were selected because they provide ideal locations for hydraulic monitoring, stable hydraulic cross sections, easy accessibility and were evenly distributed along the length of the river (Figures 3-2 and 3-3), so that reliable data would be obtained to aid in the calibration of the computational models.



Site 1 – Loch Ness



Site 2 – Englishs Road Bridge



Site 3 – Pannetts Road Bridge



Site 4 – L2 River outlet

Figure 3-2 Photographs of the capacitance water logger monitoring sites

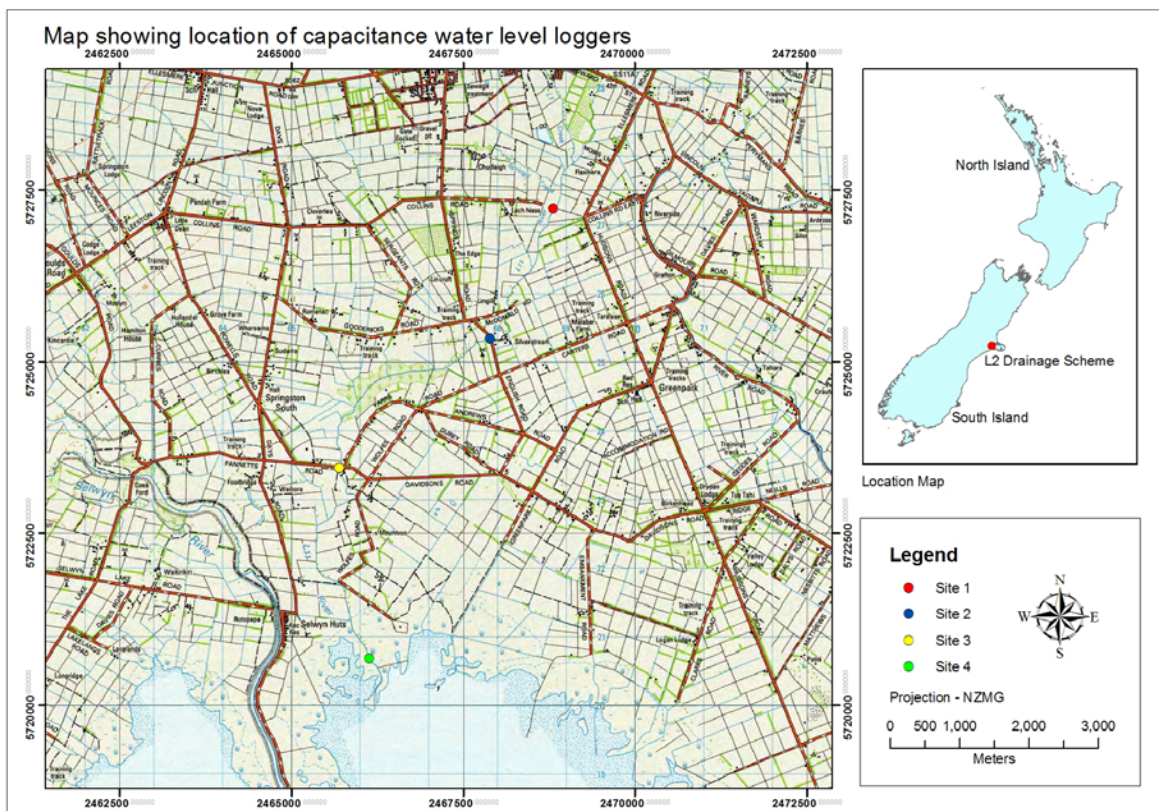


Figure 3-3 Site locations (1 through 4) (Land Information New Zealand 1:50000 Topographic Map sheets)

The length of the devices installed at the four locations varied from 0.5 m (upstream) to 1.5 m (downstream) as shown below. These lengths were selected initially based on the estimated or perceived potential variation of water level at each monitoring site.

Table 3-1 Showing the various length of probe used and location

Monitoring sites	Length (meter)
Site 1	0.5
Site 2	1.0
Site 3	1.0
Site 4	1.5

3.1.2 Calibration

Calibration of the capacitance water level probes was necessary to ensure the accuracy of measurements. Since the sensor has a linear response, it was calibrated using the two point calibration method, that is, two points were marked - one 200 mm from the bottom of the counterweight and the other equal to the length of the probe (for example, at 0.5 m for a 0.5 m length probe) (Figure 3-4).

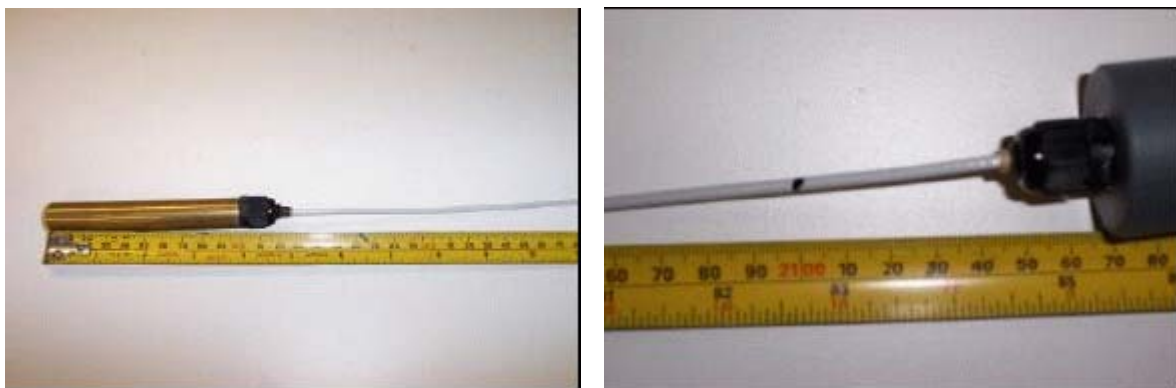


Figure 3-4 Calibration of logger (Dataflow Systems Pty Ltd.)

The following procedure was used to ensure maximum sensor measuring accuracy as recommended in the capacitance water level probe logger manual. The trace mode on a PC was used; the recorder was set to a 10 second log interval. The same heights on the probe for the two calibration values were used to obtain comparative data.

1) After the probe was cleaned (which removes any deposits that have been formed on the element), it was immersed in a water-filled bucket to the bottom mark on the probe and when the reading was stable, the value that was displayed on the computer screen was noted.

- 2) The probe was then lowered to the second point and when the reading was stable, the value that was displayed on the computer screen was noted. The probe was then removed from the bucket.
- 3) The trace mode was then aborted and the 'Enter Calibration' mode was then entered and the values obtained in steps 1 and 2 were then entered (Figure 3-5).
- 4) Steps 1 and 2 were then repeated and the values that were obtained were compared with the previous calibration. If there was a large discrepancy then the calibration was repeated. The value for the offset should be within 10 to 20 counts of the previous value.

SENSOR CALIBRATION

1000L.CAL
30300L.CAL
31411L.CAL
31413L.CAL
32254L.CAL

SENSOR CALIBRATION

SAVE TO DISK

LINEAR CALIBRATION
POLYNOMIAL CALIBRATION

SENSOR Sn. 30300 CAPACITIVE WATER LEVEL LINEAR

DATA INPUT ITEM	UNCALIBRATED VALUE	MEASURED VALUE
FIRST VALUE	748	200
SECOND VALUE	1437	500
RELATIVE VALUE	0	
NUMBER of DECIMAL PLACES	0	
CALCULATED SLOPE	2.296667	
CALCULATED OFFSET	288.6667	
UNITS OF MEASUREMENT	mm	
CLICK HERE TO CALCULATE		

PRINT CALIBRATION EXIT DONT SAVE SAVE and EXIT

Figure 3-5 Screenshot of Sensor Calibration

The formula used in the computer programme to generate calibrated data is:-

$$RES = \frac{DATA - OFF}{SL} + RL$$

Where:-

RES = Resultant calibrated water level value

SL = Slope of sensor

OFF = Offset value for sensor

RL = Relative value from bore collar to sensor position

DATA = The un-calibrated value read by the probe

Using the sensor values as shown in Figure 3-4,

The resulting calibration data are:-

$$\text{Slope} = \frac{1437-748}{500-200} = 2.29667$$

$$\text{Offset} = 748 - (200 * 2.29667) = 288.667$$

It was estimated that the capacitance water level loggers has a precision of ± 0.005 m.

3.1.3 Housing

The probe was mounted inside a vertical stilling well, which was made from Polyvinyl chloride (PVC) pipe. Holes of 6 mm diameter were drilled every 20 cm over the entire length of the shroud (Figure 3-6), therefore allowing water to freely enter the shroud. It also ensured that the water level measurements were linear over the entire length of the sensor element. The stilling well that the probe was mounted in prevented the Teflon element from touching the side of the pipe that it is mounted in and preventing water from being retained between the Teflon element and the pipe, thus preventing a measurement error at that point in the measurement range of the probe.



Figure 3-6 PVC shroud used to house the water level sensor

3.1.4 Installation

The stilling well was anchored within the channel so as to withstand the strong force of the streamflow. Waratahs were used as the anchoring mechanism and cable ties were used to tie the shroud to the waratah (Figure 3-7).



Figure 3-7 Waratah used for anchoring sensor in stream at site 1

3.2 Surveying

In hydraulic computational modelling, a channel reach is defined by cross sections and invert levels. Twelve cross-sections were used to represent the L2 River for modelling purposes. Although more cross-sections may improve accuracy, it was not technically feasible to have more.

3.2.1 Site Selection

GPS units were used to map the main channel and to identify key locations for conducting cross-section surveys. A total of 12 cross-sections between monitoring sites 1 and 4 were selected (Figure 3-8, see Appendix C for photographs of each cross section site).



Figure 3-8 Selected cross-sections along the L2 River (Land Information New Zealand 1:50000 Topographic Map sheets)

These cross-sections were selected based on stable hydraulic cross-sections, representative of a stretch of the river and where there was a noticeable change in the channel geometry. The cross sections were evenly spaced along the length of the river, so that reliable data would be obtained to aid in the calibration of the hydraulic model. A detailed survey of each cross section was conducted.

3.2.2 Surveying Methodology

The surveying procedure utilized in this research was unconventional, since direct land access to the L2 River is restricted to a few locations along the channel. The best, and possibly the only way to access the channel along most of its length and to conduct the required surveying was by using kayaks. Initially, a datum point for each cross section was put in place and marked by spray paint. It was necessary to install datum points that had an unimpeded view across the channel. This was done by driving two waratahs, one on either bank of the river, into the soil. A graduated rope (0.5 m intervals) was tied on to the datum and stretched across the channel, ensuring the rope was parallel with the water surface (Figure 3-9).

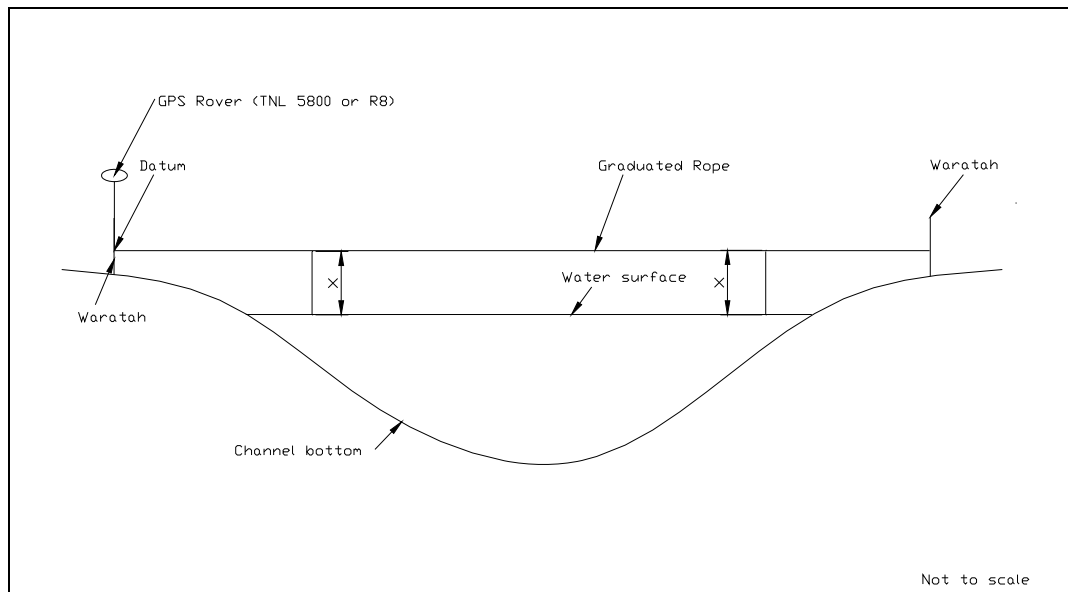


Figure 3-9 Cross-section showing surveying setup

A kayak was used to manoeuvre across the channel, to measure its depth, which was further referenced to the datum by means of the rope (Figure 3-10). The cross-sections were numbered, with number 1 starting at the upstream end.



Figure 3-10 Surveying cross sections along the L2 River using a kayak

The position and altitude of each datum point was recorded with a Trimble mapping grade surveying device with a base station, TNL 4700/5800 (or R8) RTK systems (Figure 3-11) with an estimated accuracy of ± 0.07 m. Four base station check points were established during the process (Figure 3-12) to enable a consistent measurement of elevations along the whole reach of the L2 River. It was assumed that cross-section 12 (logger # 4 at the river outlet) water levels were the same as the lake level data supplied by Environment Canterbury for a given time to derive the asl value.



Figure 3-11 TNL 4700/5800 (or R8) RTK system used to record elevation

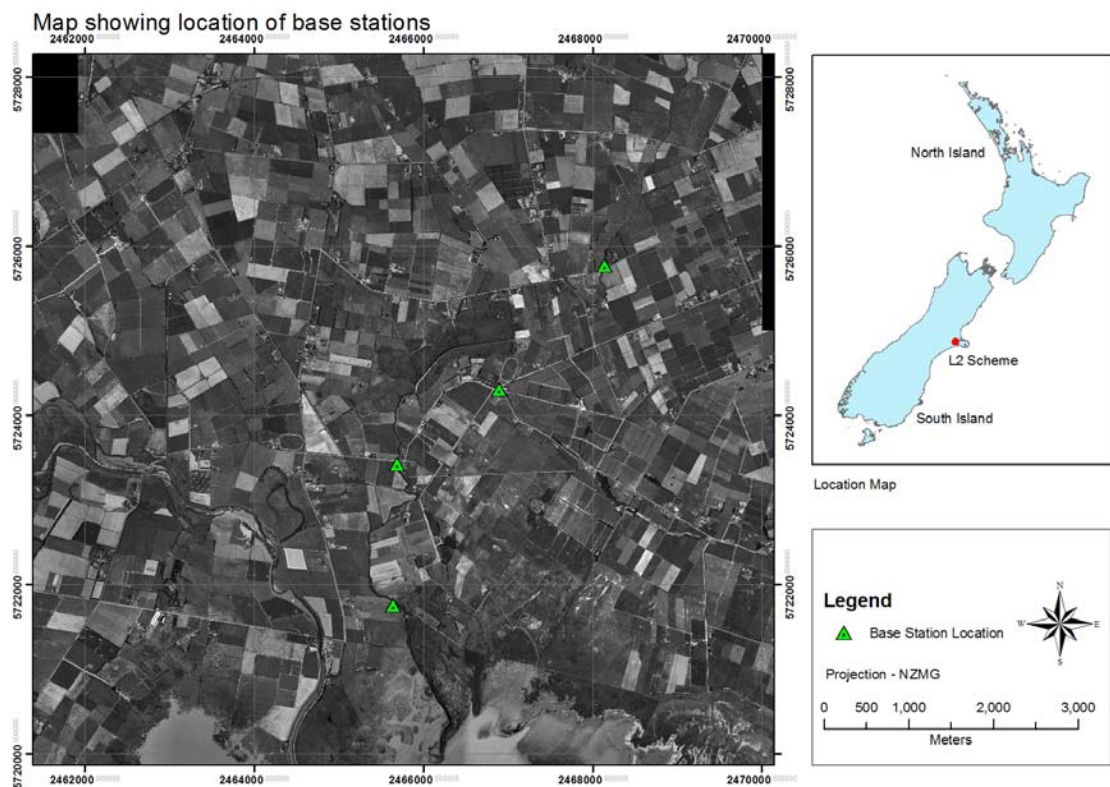


Figure 3-12 Map showing location of where base stations were established

It was difficult in some cases to determine a line perpendicular to the direction of flow when surveying cross-sections, especially near bends in the channel. After establishing the transect and datum each cross-section distance and elevation measurement (depth) were taken at approximately 0.5 m intervals and recorded on a data sheet.

To allow the cross sections to be entered into the computational model it was necessary to manipulate the survey data so that the coordinate origin was on the true left bank for all cross-sections. Correlation of surveyed invert levels and datum points were done in Microsoft Excel.

3.3 Streamflow Gauging

Streamflow gauging was done to develop a rating curve, which is the relationship between water surface elevation and discharge. A rating curve was developed for each section of the L2 stream using measurements of discharge and gauge height from the river. The discharge was used in both hydrologic and hydraulic models. The discharge was calculated using $Q=AV$ where V is the mean velocity normal to the cross-sectional area of flow and A , which is a function of the gauge height. So in order to estimate discharge, both the velocity and the gauge height must be determined. Continuous gauge heights were obtained from the capacitance water level probe loggers and water velocity was measured at key times during the study.

3.3.1 Streamflow Velocity Methodology

To measure the velocity in the L2 River, an FP101 Global Flow Probe (Figure 3-13) was used, with an estimated accuracy of ± 0.03 m/s. The speed at which the impellor rotates is proportional to the flow velocity.



Figure 3-13 FP101 Global Flow Probe

A graduated rope (0.5 m graduation) was stretched across the river for reference; hence the river was divided into 0.5 m wide subsections. A vertical velocity profile at the center of each subsection was obtained through the following methodology: the “average” function on the flow probe was “set” (zero) and then placed within the subsection, moved vertically from the surface to the bottom, up and down, slowly (Figure 3-14) and smoothly for 20-40 seconds in order to obtain a good average of the subsection velocity. The average velocity (obtained with the flow probe) times the area of the subsection equals the flow for the subsection ($Q=V \times A$). Once the flow of each subsection was obtained, all of the subsection flows were added to obtain the total streamflow.



Figure 3-14 Velocity Measurement

3.4 Roughness Coefficient (Manning's n)

The roughness coefficient (Manning's n) parameters required for Equation 2-5 (page 23) were obtained using the following procedure. Two transects 70 meters apart were established. The depth of each transect at approximately 0.5 m intervals and the water level were taken using an automatic surveying level and recorded on a data sheet. Streamflow gauging was also carried out at each transect. These data was manipulated using Microsoft Excel to calculate the roughness coefficient.

4 Analysis of Data

4.1 Lake Data

Statistics for lake openings in the period 1945 to 2005 inclusive are summarised in Tables 4-1 and 4-2. Note that an opening has been recorded only when the lake has remained opened to the sea for at least 4 days. Some ‘individual’ openings may represent many attempts.

Table 4-1 Frequency and duration of lake openings at various opening levels 1945 – 2005

Opening level (asl) (m)	No. of openings	Average days open	Average opening level (asl) (m)	Average closing level (asl) (m)
Below 1.05	18	21	0.97	0.61
1.05 – 1.2	118	20	1.12	0.66
1.21 – 1.5	58	23	1.3	0.67
above 1.5	11	33	1.63	0.63

Table 4-2 Summary of opening statistics 1945 – 2005

		Year
Max. opening level	1.89 m	1945
Min. opening level	0.85 m	1948
Max. closing level	1.48	1975
Min. closing level	0.15 m	1951
Max. openings/year	7	1975
Min. openings/year	1	'55, '71, '73, '88, '04
Av. openings/year	3.3	-

The average opening level for the summer period, since 1945, was found to be 1.14 m asl; 8.6 per cent greater than the current target level, while for the winter period the average opening level was found to be 1.24 m asl, 9.7 per cent greater than the current target opening level. The average opening level since 2000 was found to be 13.33 per cent and 10.62 per cent greater than the opening level specified by the North Canterbury Catchment Board in

1947 for the summer and winter season respectively. These higher opening levels observed, may be due to conditions that are not favourable for lake openings at the time, for example, high seas, wind from a southerly quarter and wave actions.

4.2 Hydraulic Gradient

Hydraulic gradient can be defined as the difference in head or elevation divided by the distance between two points, therefore without a gradient there would be no flow. The same principle applies to the Kaitorete Spit (Barrier): if there is no hydraulic gradient (Figure 4-1) to scour an adequate outlet, there would be no outflow to the sea.

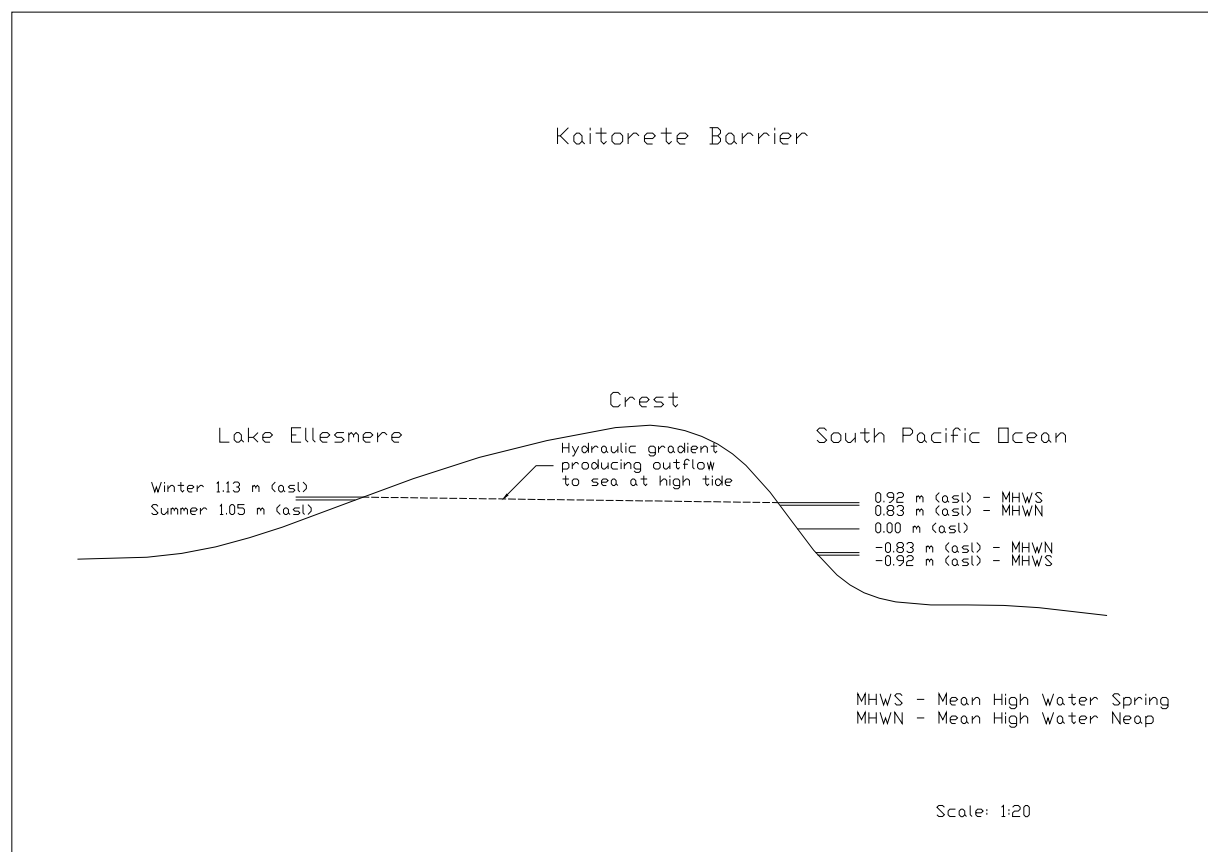


Figure 4-1 Kaitorete Barrier cross section showing relationship between lake levels and sea levels at lake opening cut to sea (Taylor 1996)

During the summer opening, the lake level is only 130 mm above the Mean High Water Spring (MHWS), while during winter this is increased to 210 mm (21 cm). With sea level rise expected to range between 9 cm to 88 cm by the end of this century (Ministry for the Environment), the hydraulic gradient will be reduced. Therefore when sea level reaches 1.05 m (asl) there will be no outflow to the sea at MHWS (no gradient). Hence in order to maintain the current gradient in any rise of sea level, it will be required that the lake level opening height be equally increased. Table 4-3 below shows the projected lake opening level (for both winter and summer) based on a 5 cm incremental increase in sea level rise.

Table 4-3 Projected Lake opening levels

Sea Level Rise (m)	Sea Level (m) (asl)	Required opening level	
		Winter (m) (asl)	Summer m (asl)
0.0	0.92	1.13	1.05
0.05	0.97	1.18	1.1
0.10	1.02	1.23	1.15
0.15	1.07	1.28	1.2
0.20	1.12	1.33	1.25
0.25	1.17	1.38	1.3
0.30	1.22	1.43	1.35
0.35	1.27	1.48	1.4
0.40	1.32	1.53	1.45
0.45	1.37	1.58	1.5
0.50	1.42	1.63	1.55
0.55	1.47	1.68	1.6
0.60	1.52	1.73	1.65
0.65	1.57	1.78	1.7
0.70	1.62	1.83	1.75
0.75	1.67	1.88	1.8
0.80	1.72	1.93	1.85
0.85	1.77	1.98	1.9

With favourable conditions, that is, wind from an easterly direction, neap tides (produces a greater outflow gradient than the spring tides), calm seas and a high lake level, an opening is easily made using bulldozers and excavators. The higher the lake level the easier it is to make a successful opening. With conditions not favourable any previous opening can be blocked in a few hours and the cut completely filled. Since it has been established that sea

rise is happening, the current opening levels will have to be increased (there will be no choice) to maintain this outflow gradient at MHWS.

4.3 L2 River Monitoring Data

Due to the fact that there were no continuous flow data available for the L2 River (Environment Canterbury only conducted flow measurement monthly at a selected location), it was determined that continuous data at multiple locations would have to be obtained. As a consequence of this, four capacitance water level loggers were installed at strategic locations (as mentioned in section 3.1.1). Data were recorded every 10 minutes and analysed for the period 25 July to 31 December 2006.

4.3.1 Water Level Loggers

Data from the loggers were calibrated and plotted on a monthly basis, also in conjunction with the streamflow data; rating curves were created for sites 1, 2, 3 and 4 to calculate flow rate which was later used in the modelling process. After logging for the initial period from 25 July 2006 to 18 August 2006, it was found that the loggers at sites 1 and 3 were set too low in the river; the peaks of rainstorm events were not logged (loss of data), hence the flattened effect of the peaks, as shown in Figure 4-2. Table 4-4 shows the maximum and minimum water level each sensor can record within the L2 channel.

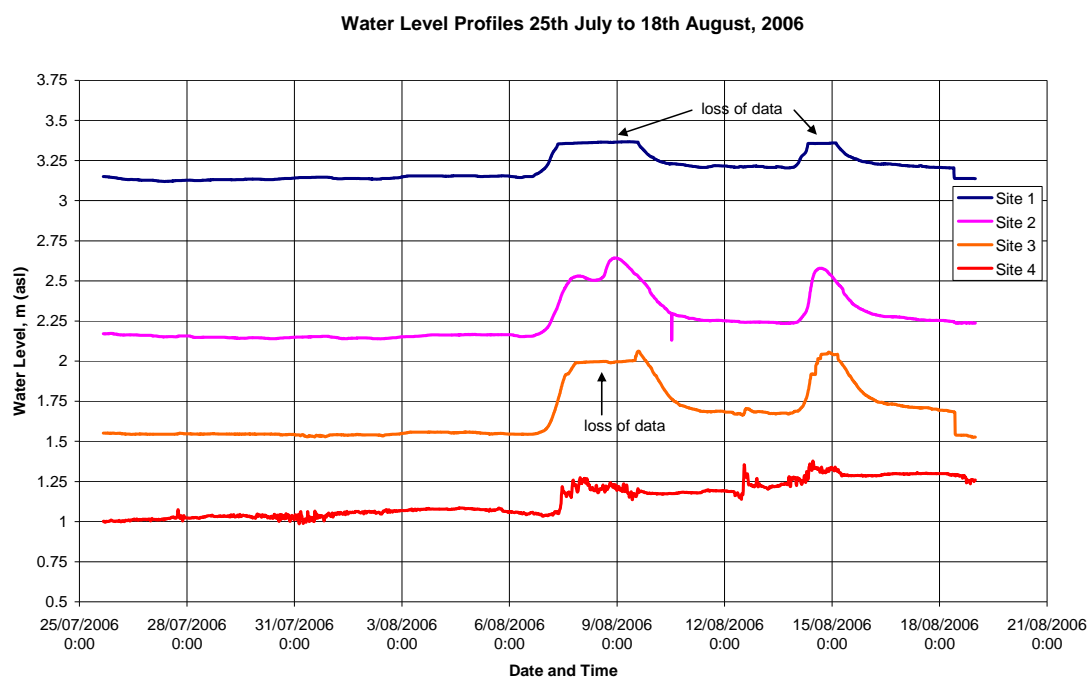


Figure 4-2 Recorded water levels at sites 1, 2, 3 and 4 for the period of 25th July to 18th August, 2006

Table 4-4 Water level limits for each sensor

Sensor location	Sensor length (m)	Min. water level (asl) (m)	Max. water level (asl) (m)
Site 1	0.500	2.970	3.370
Site 2	1.000	1.821	2.721
Site 3	1.000	1.149	2.049
Site 4	1.500	0.410	1.810

A decision was made to raise the loggers at sites 1 and 3 by 65 mm and 139 mm respectively, which was done on 18 August, 2006. (However, site 1 was not able to capture peak flows for extreme events, thus the logger was deemed too short in length.)

Extreme events are clearly identified in Figures 4-3 and 4 for sites 1, 2, and 3. However site 4 logger (closest to Lake Ellesmere) showed minor variation in water level from July to

August (Figure 4-3), with major variations within the period September to December (Figure 4-4), even when extreme events were not recorded in sites 1, 2 and 3.

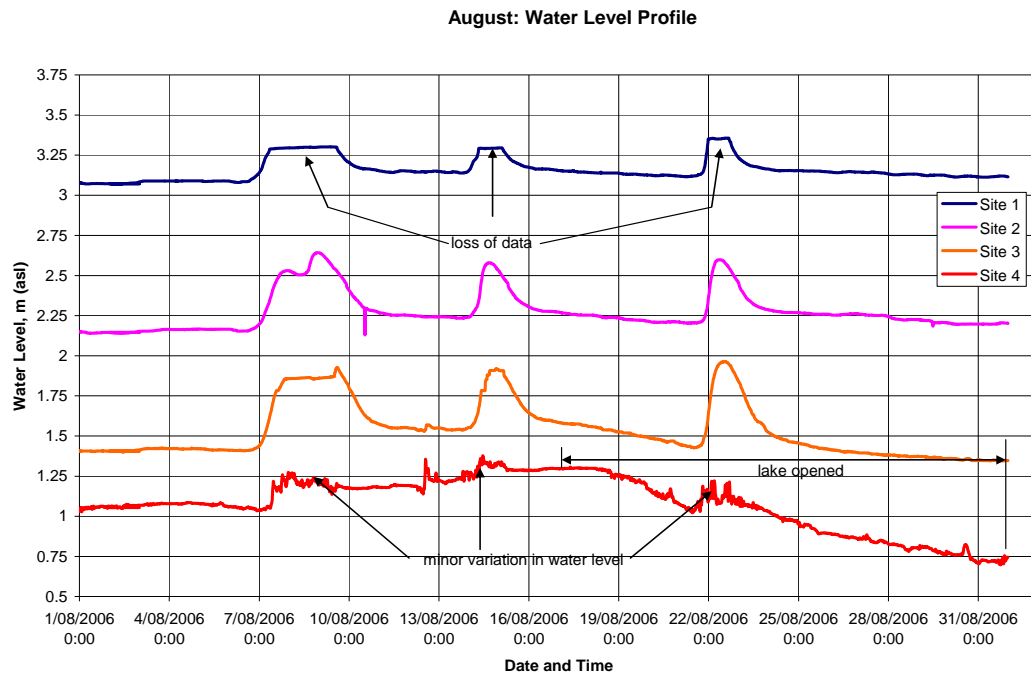


Figure 4-3 Minor variation in water level at site 4

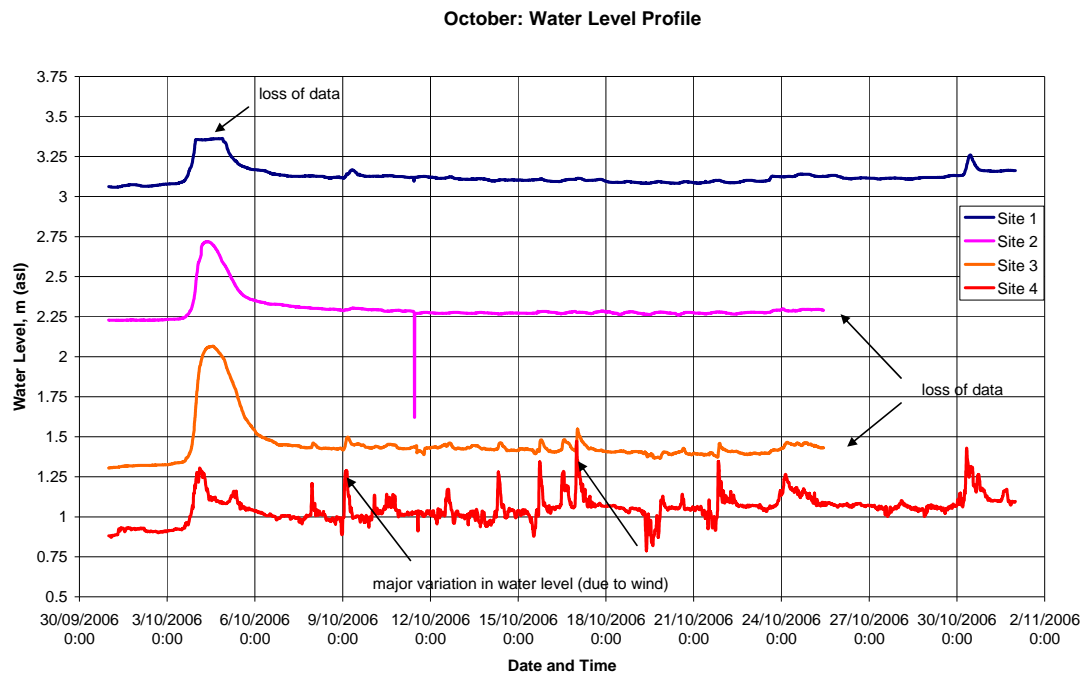


Figure 4-4 Major variation in Site 4 water level

This rapid variation in lake levels was due mainly to strong winds acting on the large surface area of the lake. As Lake Ellesmere is a shallow lake, wave length and wave height are reduced, in contrast to deeper water of equivalent fetch. Because of the shallowness of Lake Ellesmere, it results in a tilting of the water surface known as set-up (Taylor 1996). Figure 4-5 and Figure 4-6 show waves moving upstream in the L2 River at site 4, caused by a southeasterly wind.

An increase in water level at the mouth of the L2 River by 47.58 per cent on the 21 October 2006, only resulted in an increase in water level at site 3 by 5.73 per cent. Thus site 3 is only being affected to a very small extent by the wind.

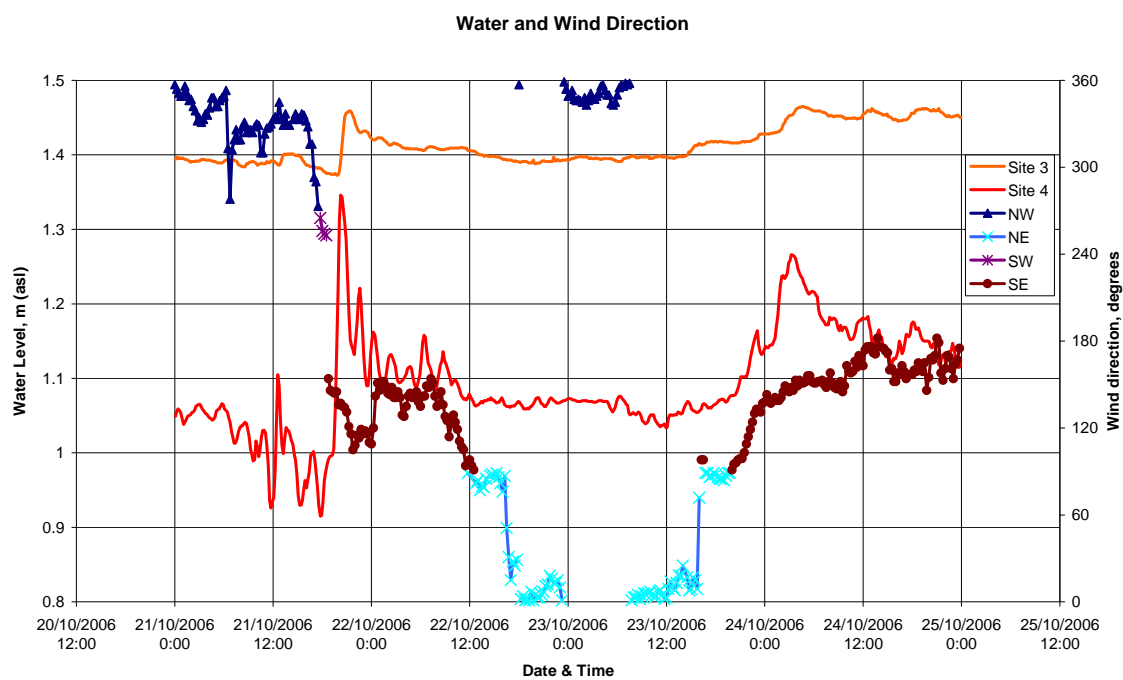


Figure 4-5 Correlation of the wind direction and water level for the period of 21 to 24 October 2006 (wind data provided by Environment Canterbury from their hydrotel site within Lake Ellesmere).



Figure 4-6 Wave moving upstream, due to wind (2006-12-7, 17:40 hrs)

As seen in the example provided in Figure 4-5, a wind from the north-westerly sector causes significant reduction of water at the mouth of the L2 River (site 4). A south-east wind causes an accumulation of water at the mouth of the river. As mentioned by Taylor (1996) a south-westerly wind can raise the lake level on the northern side of the lake by more than 0.6 m. However, it is shown here that south-easterly and not the south-westerly wind has this effect on the L2 River. This is because the Selwyn River outlet peninsula (Figure 4-7), which is located within close proximity west of the L2 River, extends further into Lake Ellesmere than the L2 River, thus protecting the L2 River from south-westerly winds.

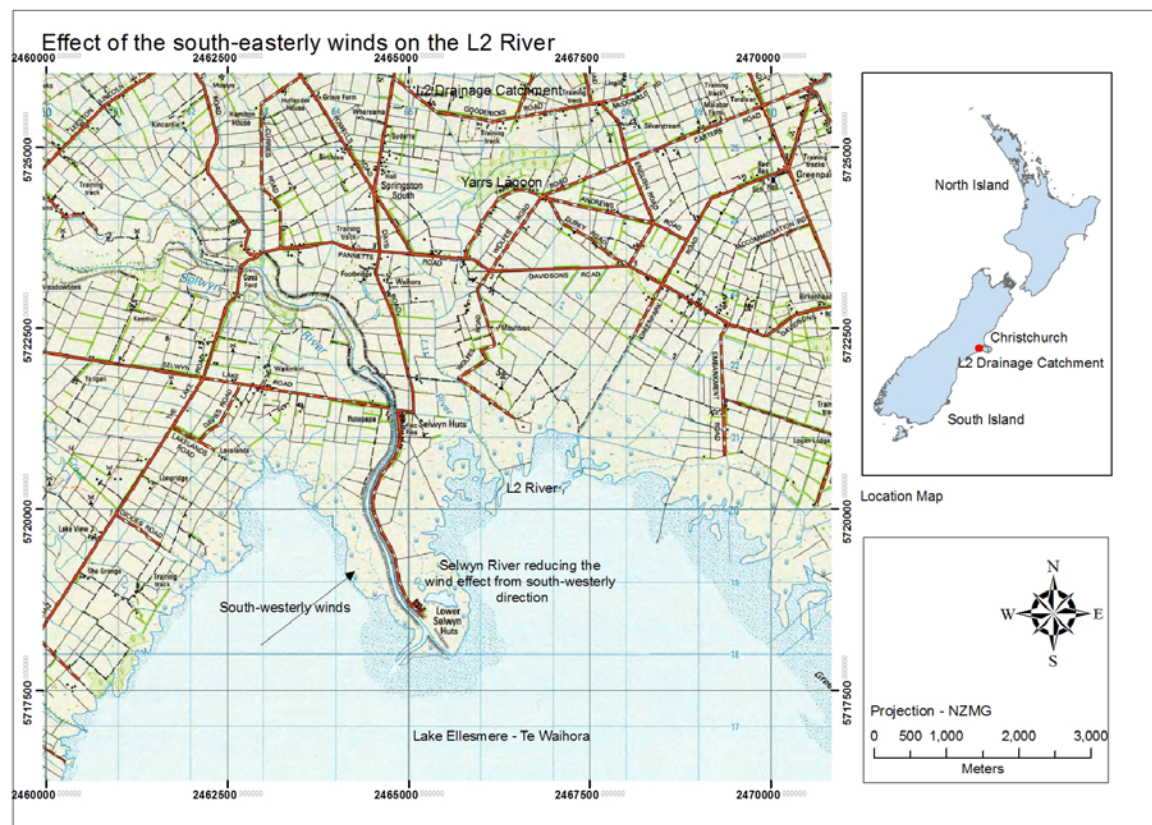


Figure 4-7 Effect of south-easterly winds on the L2 River (Land Information New Zealand 1:50000 Topographic Map sheets)

Wind induced “set-ups” result in occasionally significant inundation of the surrounding habitats/farmland, as shown in Figure 4-8. Stock has been drowned as a result of these wind-induced changes in the lake level (Taylor 1996).



Figure 4-8 Inundation of surrounding habitats

One factor that had a minor effect on the accuracy of the capacitance water level logger within the channel is the floating weeds that would normally “get stuck” on the housing and support of the loggers (Figure 4-9). This was very prevalent at sites 2 and 3, but has only caused the measured water level to rise a few mm higher than the actual water level (Table 4-5). The table shows that weeds have caused a 3 to 4 mm difference in the water height.



Figure 4-9 Weed stuck to the logger housing and support (2006-08-10), site 3

Table 4-5 Comparison of water levels before and after the weeds have been removed

Site 2

	Date	Time	Water Level m (asl)
Before	18/08/2006	9:55:20 a.m.	2.244
	18/08/2006	10:05:20 a.m.	2.243
After	18/08/2006	10:05:48 a.m.	2.240
	18/08/2006	10:15:48 a.m.	2.240

Site 3

	Date	Time	Water Level m (asl)
Before	10/08/2006	1:10:00 p.m.	1.761
	10/08/2006	1:20:00 p.m.	1.761
After	10/08/2006	1:27:14 p.m.	1.757
	10/08/2006	1:37:14 p.m.	1.756

4.4 Surveying Results and Discussion

The deepest level of the invert of each cross section was plotted against the chainage (longitudinal length of the river) to give a long section from site 1 to the outlet (Lake Ellesmere), shown below in Figure 4-9. There was a significant difference in slope between each section.

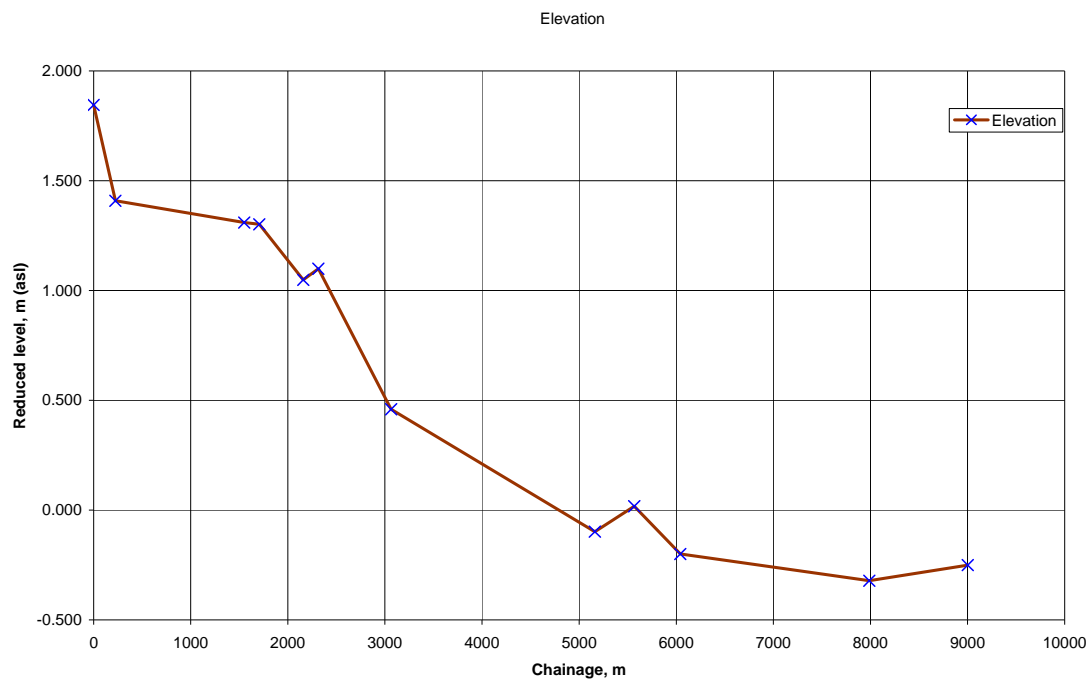


Figure 4-10 Deepest point of each cross section along the channel from site 1 to outlet (Lake Ellesmere), and the reaches along the river (between surveyed points)

The mean bed slope (deepest points), based on the total fall and total chainage between cross section 1 and cross section 12, was 1 in 4500 or 0.0002. To give an indication of the degree of variability in the invert slope (deepest point) along the channel, the slope within each section was calculated as a function of chainage (Table 4-6). Generally the slope became flatter closer to the lake. Table 4-5 shows the water surface slope measured for both the winter and summer periods using data recorded from the water level loggers, at the four

monitoring sites. The lower reaches of the river for the summer period showed a greater water surface slope than for the winter period. This may be due to the lower lake level observed for the corresponding period.

Table 4-6 Length and slope for each reach of the river surveyed

	Length m	Deepest Point			Water Surface slope	
		Slope m/m	Elevation start m (asl)	Elevation end m (asl)	Winter water level slope m/m	Summer water level slopes m/m
Reach 1	225.25	0.0019	1.845	1.409	0.0004	0.0004
Reach 2	1326.07	0.0001	1.409	1.310	0.0004	0.0004
Reach 3	155.43	0.0001	1.310	1.302	0.0004	0.0004
Reach 4	452.33	0.0006	1.302	1.049	0.0004	0.0004
Reach 5	154.72	-0.0003	1.049	1.099	0.0004	0.0004
Reach 6	750.51	0.0009	1.099	0.459	0.0002	0.0003
Reach 7	2097.8	0.0003	0.459	-0.098	0.0002	0.0003
Reach 8	404.79	-0.0003	-0.098	0.017	0.0002	0.0003
Reach 9	474.48	0.0005	0.017	-0.200	0.0001	0.0002
Reach 10	1947.85	0.0001	-0.200	-0.322	0.0001	0.0002
Reach 11	1012.83	-0.0001	-0.322	-0.250	0.0001	0.0002

Figure 4-11 shows the water surface slope observed for both winter and summer periods. Because data from the loggers were used to calculate this slope, the slope for the reaches found between the sites was assumed to be the same, for example, for all the reaches located between sites 1 and 2, the water level slope was assumed to be constant.

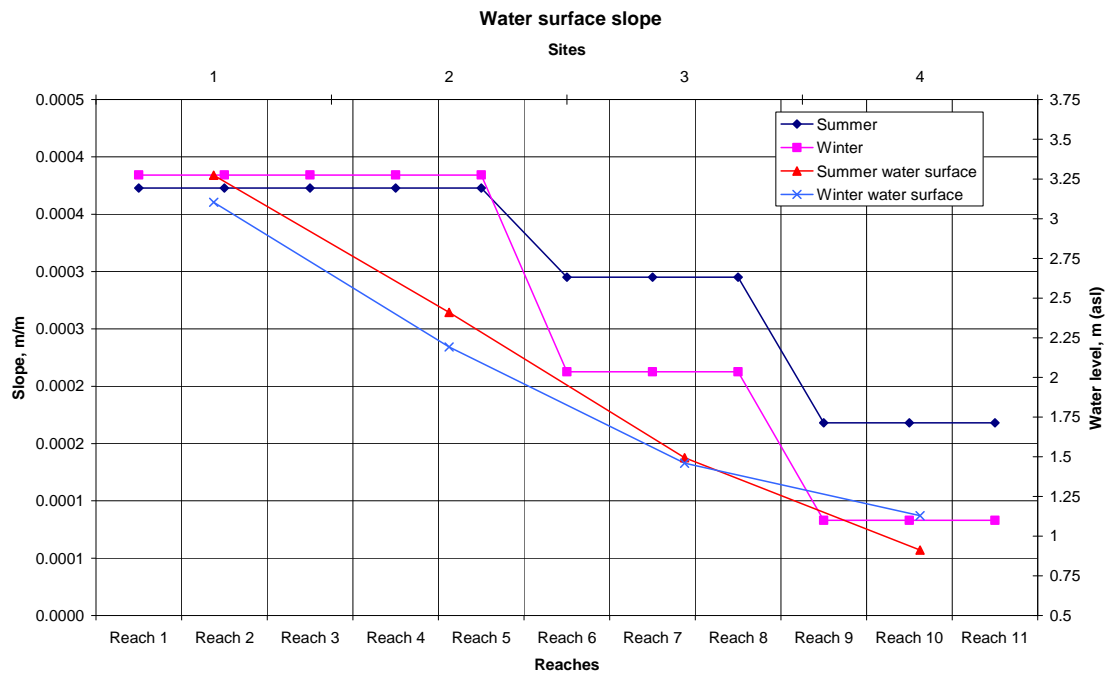


Figure 4-11 Slopes and observed water surface level for both winter and summer periods

The slopes of the channel bottom at bridges 2 and 3 (chainages 23+13.8 (reach 5) and 55+66.9 m (reach 8) respectively) were found to be negative (Table 4-6) as well as the outlet. A negative slope occurs when bed elevation is higher at the lower end of a reach. Two possible causes of negative slope within the channel are construction waste (from the construction of the bridge) and sedimentation. Whenever the river is de-silted, it is only done between the bridges and not under, hence the negative slope. The deepest section of the river was found to be between chainage 30+64.31 and 51+62.11 m (Yarrs Lagoon) and as a result of this, ponding takes place. In periods of high flows, in the event that the section is overtopped Yarrs Lagoon (also known as the “big sponge”) acts as a reservoir.

All cross-sections were plotted in Figure 4-12, with the reduced level of the invert slope on the vertical axis. This figure also illustrates the variability of the channel bed slope to some

extent, with the cross sections overlapping and crossing in a number of places, due to the bends within the L2 River and variability of the terrain.

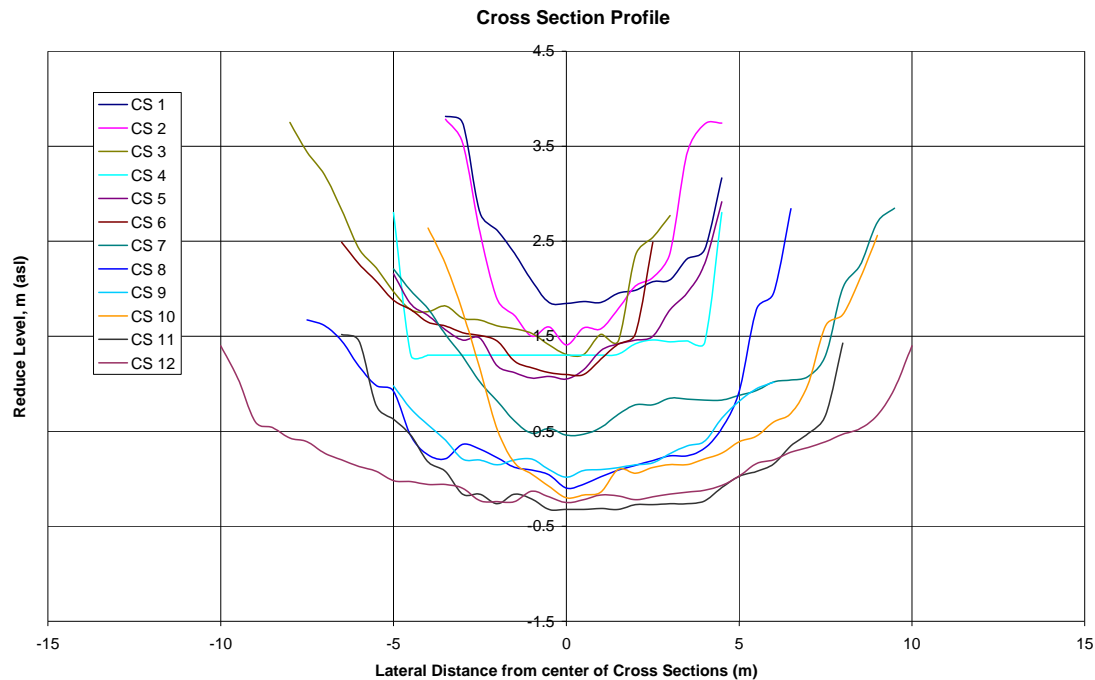


Figure 4-12 Cross-section profiles

The main factor that affected the accuracy of the surveyed cross sections for this project was the weeds found at the bottom of the channel. The weeds act as a “false” bottom and interacted with the weight of the staff coupled with the force exerted on the staff used for measuring the cross-sections. Another potential source of error is that the channel bottom is composed of soft clay (sediments); hence an incorrect depth of the channel may be recorded. To overcome this problem, the same person conducted all cross-sections and tried to exert the same pressure every time a measurement was taken. It is difficult to give an estimation of precision for surveying of the cross sections using a surveying staff; however it was estimated to be ± 0.150 m.

4.5 Streamflow Gauging Results and Discussion

The water level data obtained together with velocity measurements taken over a period of time were used to create rating curves for all four monitoring sites. The velocity measurements were conducted over a period of time to create the curve for each site. The mean velocity profiles of the four sites are shown in Figures 4-13, 14, 15 and 16.

The shape of the velocity plot in Figure 4-15 is different from the others. Upon further investigation, it was discovered that a clump of weed was located approximately 10 m upstream in the middle of the channel. This diverted the channel flow around the weeds, effectively reducing the cross sectional area and increasing the water velocity at the sides of the channel, hence the shape of the velocity profile.

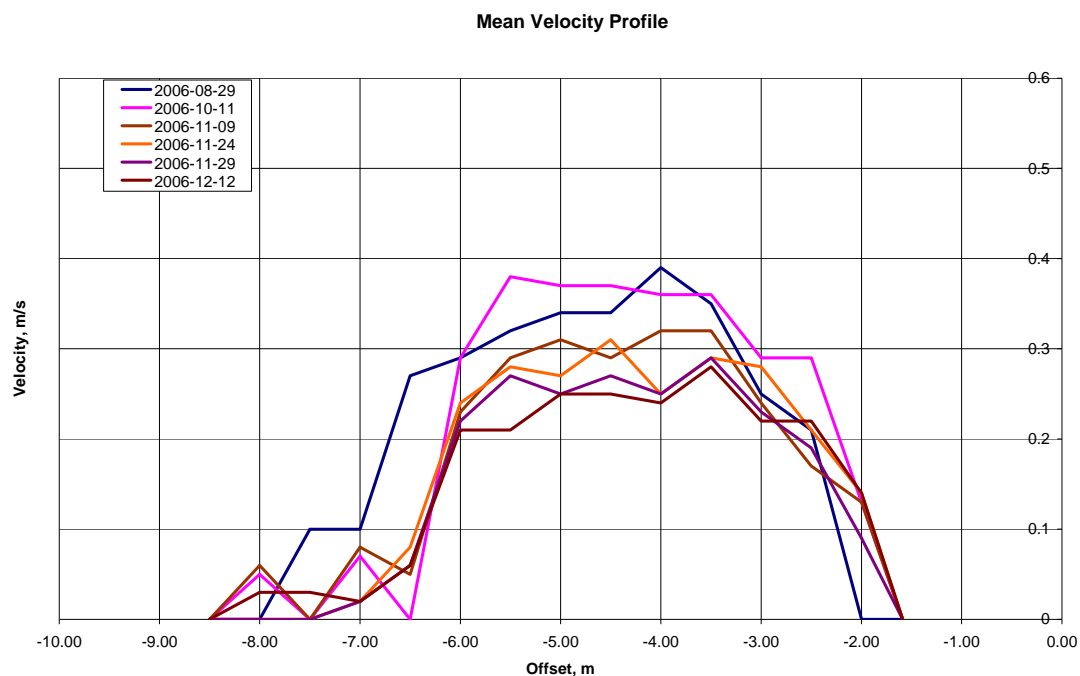


Figure 4-13 Measured velocity profiles, site 1

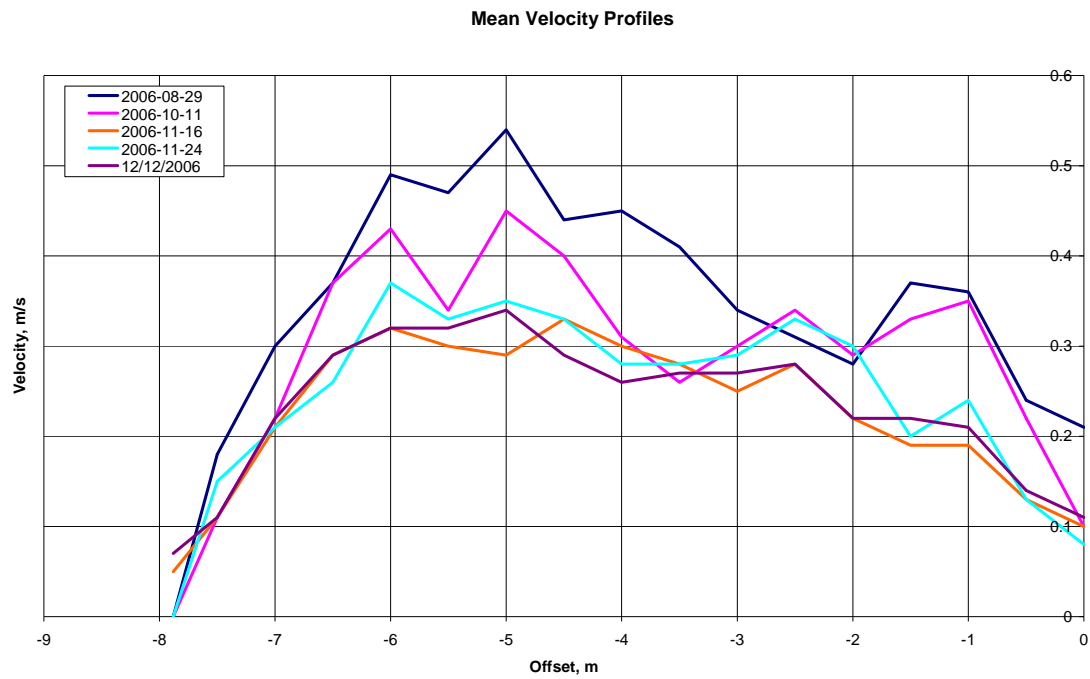


Figure 4-14 Measured velocity profiles, site 2

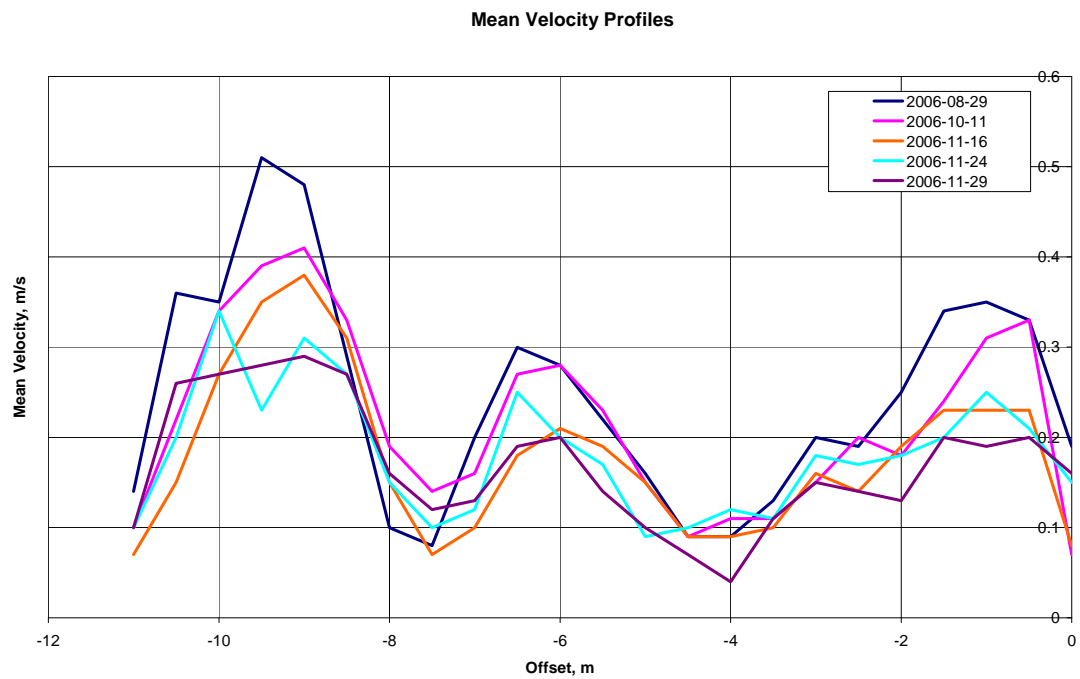


Figure 4-15 Measured velocity profiles, site 3

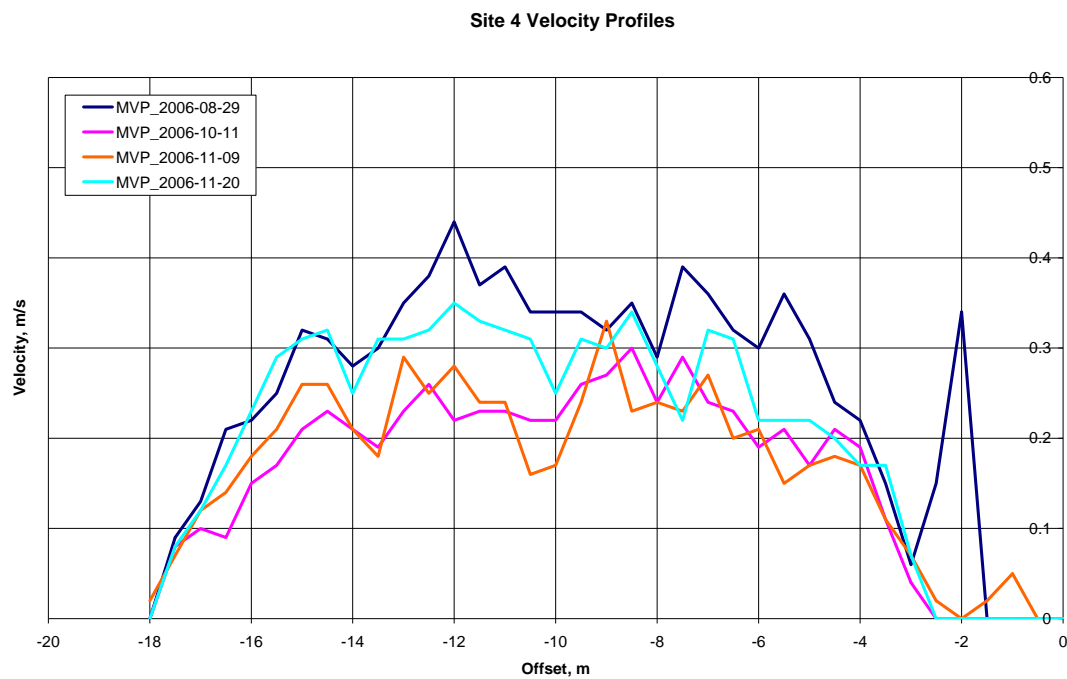


Figure 4-16 Measured velocity profiles, site 4

The variation in the velocity within the profile curves was due to weeds within the channel. As the weed grows (mid October to December) within the channel, it effectively reduces the cross-sectional area of the channel and hence causes the water to rise within the channel over the time (Table 4-7), especially at site 1 (Figure 4-17). The water level at site 1 continued to rise within the channel even though the water level at the outlet (site 4) was reducing; the lake was opened to the sea, hence the reduction in water level at site 4 for the same corresponding period.

Table 4-7 Reduction of velocity coupled with a rise in water level due to weed growth**Site 1**

Date	Mean Channel Velocity m/s	Level m (asl)
29/08/2006	0.21	3.125
11/10/2006	0.21	3.119
9/11/2006	0.18	3.216
24/11/2006	0.17	3.234
29/11/2006	0.15	3.247
12/12/2006	0.14	3.323

Site 2

Date	Mean Channel Velocity m/s	Level m (asl)
29/08/2006	0.36	2.209
11/10/2006	0.30	2.275
16/11/2006	0.23	2.317
24/11/2006	0.24	2.367
12/12/2006	0.23	2.400

Site 3

Date	Mean Channel Velocity m/s	Level m (asl)
29/08/2006	0.24	1.365
11/10/2006	0.22	1.440
16/11/2006	0.18	1.368
24/11/2006	0.18	1.392
29/11/2006	0.17	1.378

Site 4

Date	Mean Channel Velocity m/s	Level m (asl)
29/08/2006	0.26	0.780
11/10/2006	0.17	1.028
9/11/2006	0.18	1.115
20/11/2006	0.22	0.761

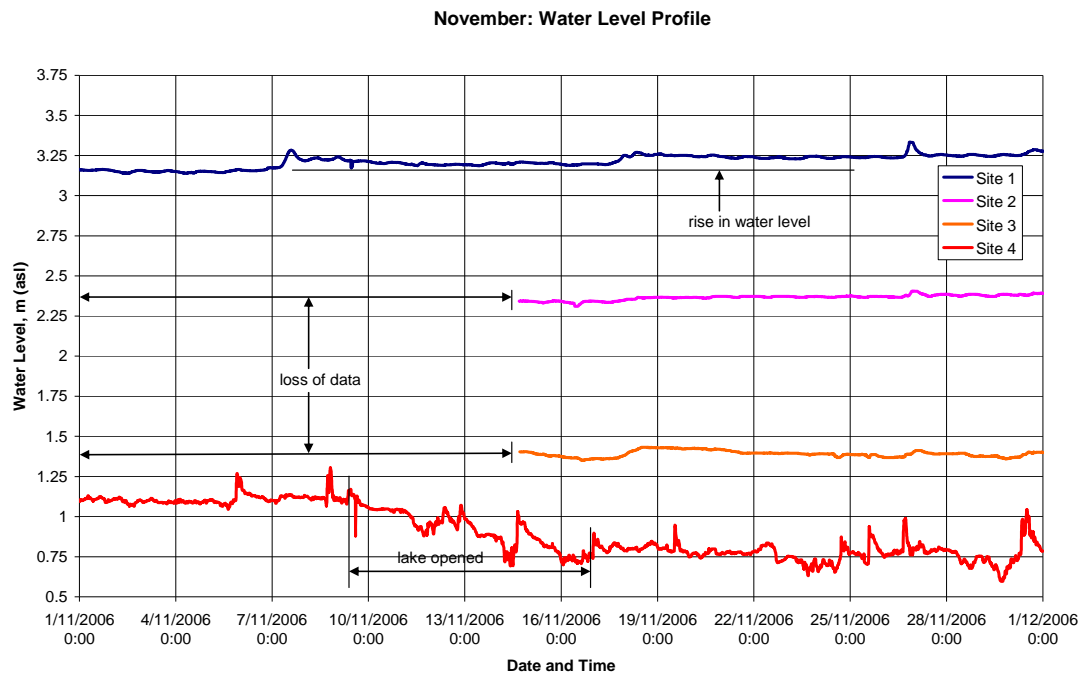


Figure 4-17 Water Level Profile for the month of November

As a result of the rapid growth of the vegetation within the channel, the shape as well as the accuracy of the rating curves was affected. Since the water level is high within the channel and the velocity is low, this resulted in the discharge being less than anticipated (Figure 4-17). To overcome this issue, two rating curves were created for each site (July to October and October to December) using the data measured for each period. These curves were also created based on the knowledge and understanding of the flow observed within the channel, from time spent in the field and they are the best curves possible.

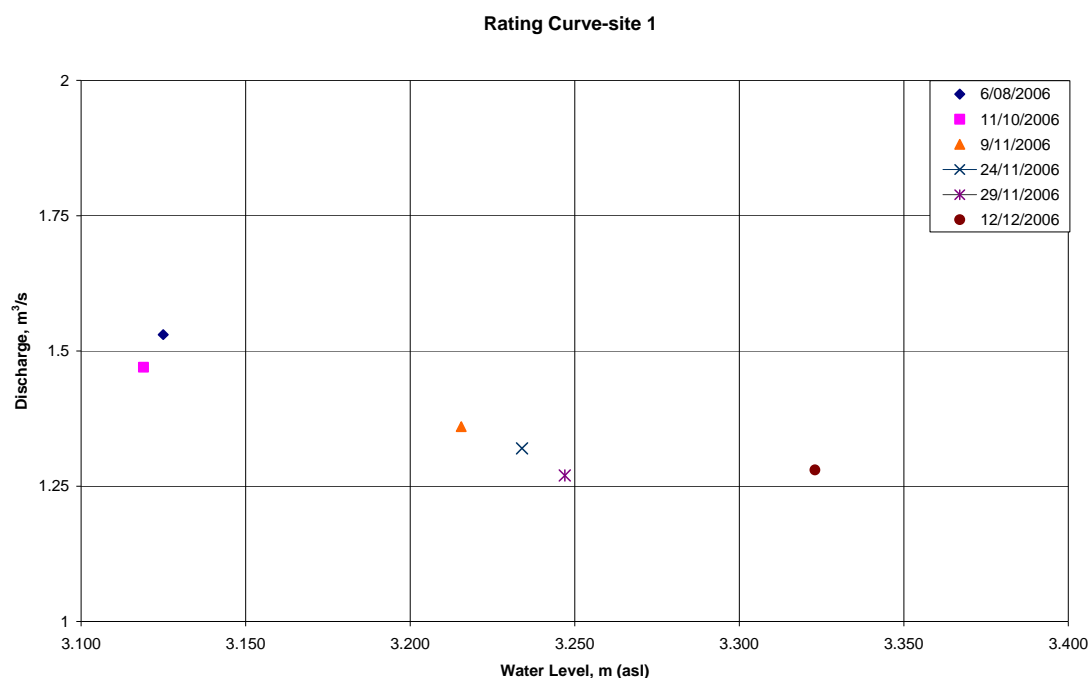


Figure 4-18 Relationship of discharge vs water level at different periods for site 1

Table 4-8 Rating curves equation

Location	Jul - Mid Oct	Mid Oct - Dec
Site 1	$0.8753x - 1.2331$	$2.6104x - 6.6409$
Site 2	$1.4507x - 1.2167$	$6.9721x - 14.655$
Site 3	$1.8447x - 0.1667$	$3.2837x - 2.6423$
Site 4	$2.1057x + 1.1158$	$1.0527x + 2.311$

One potential error in measuring the velocity that had to be overcome was the wind. When the probe is taken out of the water and the wind is strong (depending on the direction) the propeller continues to spin, thus requiring caution when removing the probe from the water to read the average velocity for the subsection. Another potential error was the weed that was present within the subsection. Whenever the propeller was placed into the channel and there was weed at the channel bottom (sometimes you cannot see the river bed), the propeller stops spinning, therefore slightly affecting the average velocity reading for that subsection.

4.6 Rainfall Data

Most floods throughout the Lake Ellesmere catchment result from heavy rainfall during south-westerly conditions (Taylor 1996). The effect of rainfall within the L2 catchment is very prevalent during the winter season. A major rainstorm will cause water levels to rise significantly within the L2 River (Figure 4-19). Figure 4-19 shows the daily rainfall data for the Broadfield EWS station in Lincoln (H32645) with coordinates -43.62622, 172.4704 (latitude (dec.deg), longitude (dec.deg)) and water level recorded by the water level loggers for the month of August 2006. The amount of water level increase within the channel will vary depending on the intensity of the rainstorm as well as the duration of the storm.

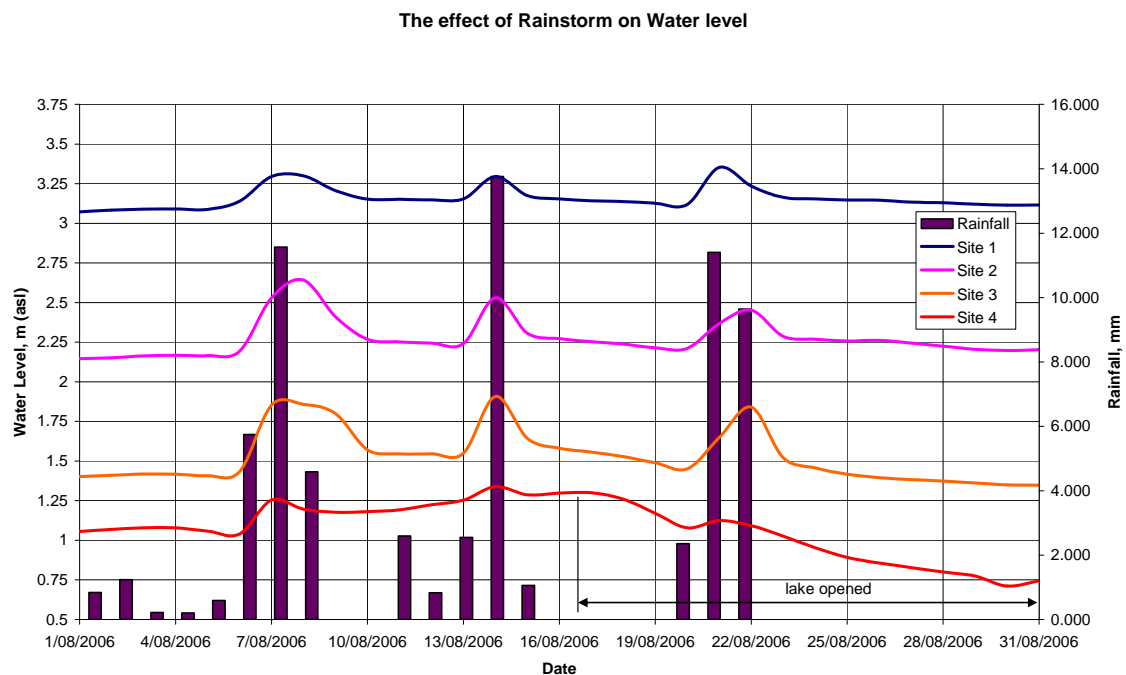


Figure 4-19 The effect of Rainfall for the month of August

As a result of the high water level within the channel, overtopping of the banks can occur along the length of the river, resulting in ponding/flooding. Rainfall data together with the

water level data from all four sites were used to estimate the lag time and time to peak.

These data were very important for the calibration of the hydrologic model.

5 Computational Modelling

5.1 Hydrological Model

5.1.1 Introduction

The Hydrologic Modelling System (HEC-HMS) was designed and developed by the US Army Corps of Engineers, to simulate the precipitation-runoff processes of dendritic (branch-like) catchment systems. It was designed to be applicable in a wide range of geographic areas for solving the widest possible range of problems, which includes large river basin water supply and flood hydrology, and small urban or natural catchment runoff. Hydrographs produced by this program can be used directly or in conjunction with other software (in this case MIKE 11) for studies of water availability, urban drainage, flow forecasting, future urbanization impact, reservoir spillway design, flood damage reduction, floodplain regulation, and systems operation. The HEC-HMS model was selected because it incorporates the SCS loss, lag time routing (both hydrologic and hydraulic) and other hydrologic methods for which data were available. Other reasons for selecting HEC-HMS were that the software is freely available and it has a user-friendly interface.

The HEC-HMS model consists of three components: the meteorological model (climatological data), the basin model (element data and connectivity) and the control specification (simulation duration and time step).

5.1.2 Meteorological Model

The meteorological model provides for meteorologic data representation as well as data analysis and includes precipitation, evapotranspiration and snowmelt. For this study, only precipitation was used, since evaporation was minimal within the catchment and is also

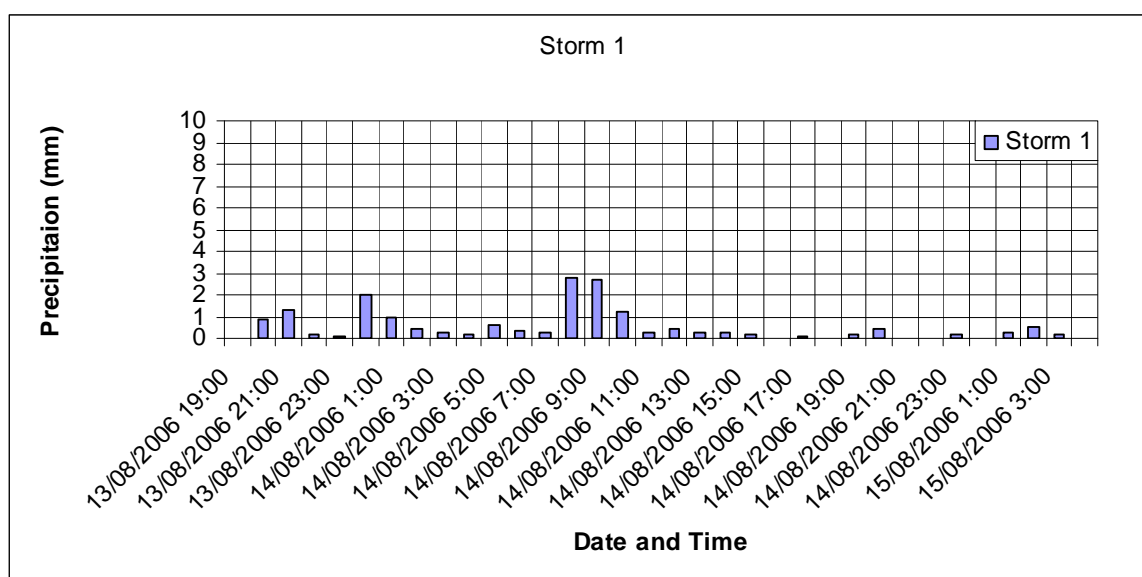
included within the losses calculation. Six different historical and synthetic precipitation methods were available, but only the specified hyetograph method was used.

5.1.2.1 Precipitation

The response of a catchment is driven by precipitation that falls on the catchment and evapotranspiration from the catchment. The precipitation inputted into the model was in the form of observed rainfall from a historical event. For this study, historical precipitation data were obtained from the National Institute of Water and Atmospheric Research (NIWA), for the Broadfield EWS station in Lincoln (H32645) with coordinates -43.62622, 172.4704 (latitude (dec.deg), longitude (dec.deg)) and was used for the calibration and verification of the model. For evaluating the performance of the L2 catchment, *hypothetical* or *design storms* were used to describe future risk of flooding.

5.1.2.2 User specified hyetograph

A number of storms were used for the calibration and verification of the hydrological model for the period of August to December 2006, but only three are presented in this report since they represent the range of variations in both intensity and duration (Figure 5-1).



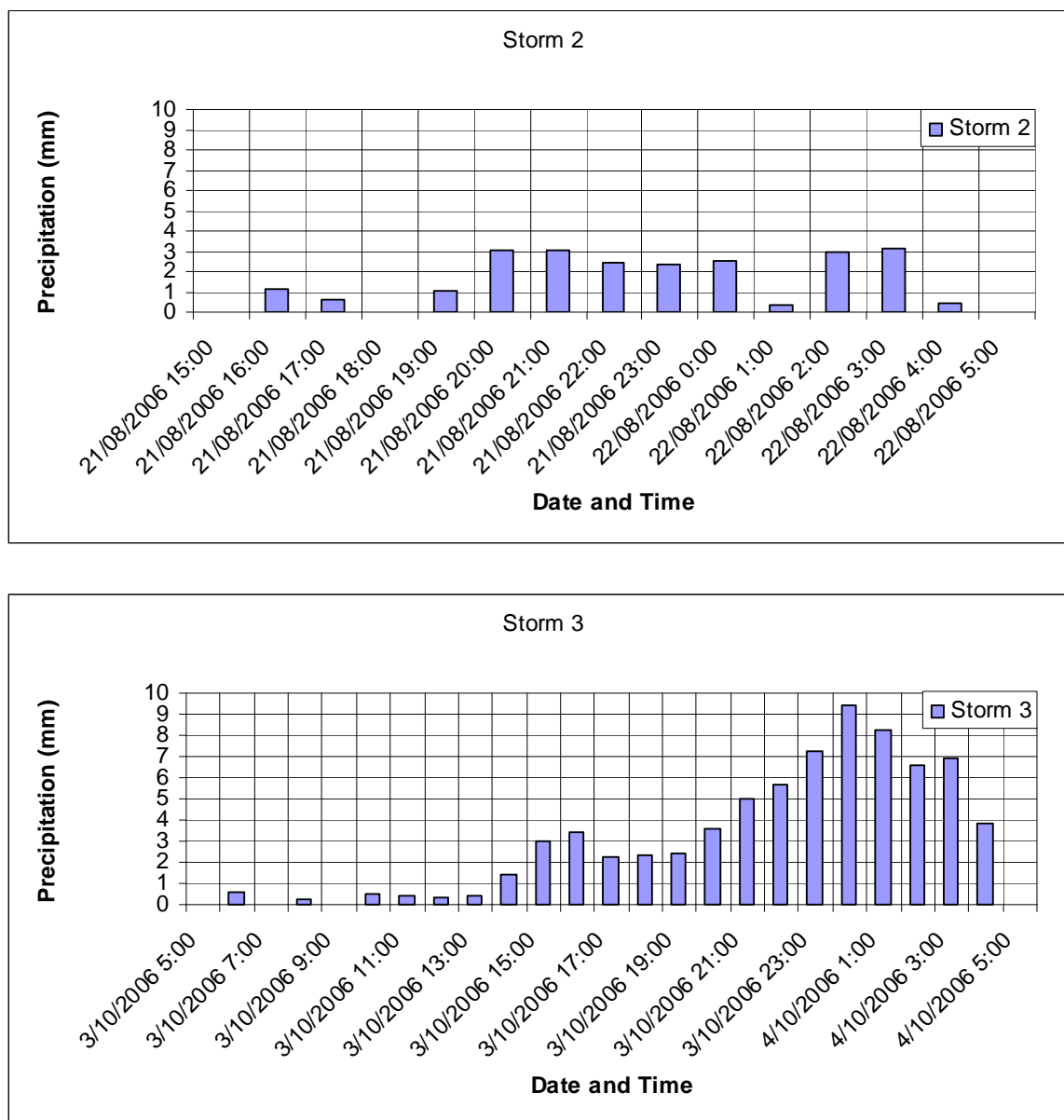


Figure 5-1 Three examples of the storm hyetographs used for calibrating the hydrological model

5.1.3 Basin Model

The basin model was used to define the physical representation of the L2 catchment. The hydraulic elements (sub-basin, reach and junction) were connected in a dendritic network (schematic) to simulate the runoff process (Figure 5-2).

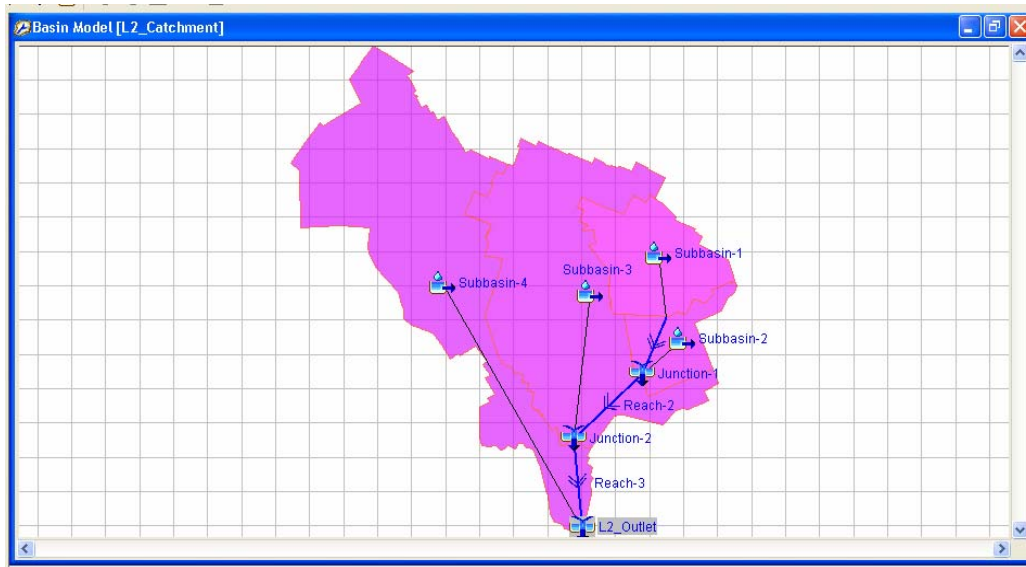


Figure 5-2 Physical representation of the L2 catchment showing subbasins, reaches and junctions

The HEC-Geospatial Hydrologic Modelling Extension (GeoHMS) tool normally used to analyse digital terrain information and develop a number of hydrologic modelling inputs was unable to be used to define the L2 catchment. This is because the digital elevation model (DEM) available was of a low resolution (25 m), which impeded automatic delineation of a catchment in the flat terrain of the scheme. As a result of this, aerial photographs along with a drain map (GIS layer) of the area were used to visually define the catchment using ArcGIS. The hydrologic structure was manually defined using the required elements (Figure 5-2).

The computation for a simulation proceeds from the upstream elements in a downstream direction. The basin model was also used to represent the hydrologic process, that is, precipitation losses, transform (surface runoff), baseflow and hydrologic routing.

5.1.3.1 Precipitation Losses

HEC-HMS computes runoff volume by computing the volume of water that was intercepted, infiltrated, stored, evaporated, or transpired and subtracts it from the precipitation.

Interception and surface storage were intended to represent the surface storage of water by trees or grass, local depressions in the ground surface, cracks and crevices in parking lots or roofs, or a surface area where water was not free to move as overland flow. Infiltration represents the movement of water to areas beneath the land surface. Interception, infiltration, storage, evaporation, and transpiration collectively are referred as *losses*. There were nine different loss models that could be selected from within HEC-HMS.

The SCS curve number loss model was used in this research, since it is the most widely used method and also because of data availability. HEC-HMS computes incremental precipitation during a storm by recalculating the infiltration volume at the end of each time interval. Three variables that were needed for the computation of losses are: Initial Abstraction, I_a ; Curve Number, CN; and Impervious.

5.1.3.2 Initial abstraction

The initial abstraction defines the amount of rain that must fall before rainfall excess results (Scharffenberg and Fleming 2006). In changing the initial abstraction, this changes the infiltration response later in the storm. If this value was left blank, it was automatically calculated as 0.2 times the potential retention, S , which was calculated from the curve number (Equation 2-2). Table 5-1 shows the initial abstraction parameter used in the loss model to calculate the losses. This value was estimated from a typical depression retention for various land covers table.

Table 5-1 Initial abstraction used in the SCS curve number loss model

Subbasin	Initial Abstraction (mm)
1	12.7
2	12.7
3	12.7
4	12.7

5.1.3.3 Curve Number

The curve number is a function of land use, antecedent soil moisture and hydrologic soil group within a catchment. The curve number is a dimensionless number defined such that $0 \leq CN \leq 100$. For impervious and water surfaces $CN = 100$; for natural surfaces $CN < 100$.

The curve numbers for the L2 catchment were derived utilising ArcMap 9, using the land use (obtained from the New Zealand land use cover database) and soils layers (obtained from the New Zealand Fundamental Soils Layer – FSL). All the features in the drain_class (drainage class) field from the soils layer, were “dissolved” to obtain a map with only the drain_class field. The attribute table was then edited to add a new field: hydr_soil (hydrologic soil). Using the drain_class field, their respective soil type was then assigned (Table 5-2) and the soils map was then “dissolved” again to show the hydrologic soil types (A, B, C & D). To simplify the landuse map, the lcbd1name field was “dissolved” map.

Table 5-2 Conversion of drain_class to hydr_soil

drain_class	hydr_soil
1	D
2	D
3	C
4	B
5	A
B	D
l	D
r	D
t	D

The hydrologic soil group and the landuse map were then combined using the GIS “union” tool. The curve number for each combination of soil type and land use was then estimated from a runoff curve numbers table (see Appendix D for the table) and entered; the average

curve number for each sub-basin was then obtained. Table 5-3 shows the curve numbers used in the SCS loss model to calculate the losses.

Table 5-3 Curve number used in the SCS curve number loss model

Subbasin	Area (m ²)	Curve Number
1	10043100	74
2	5000000	78
3	30899400	53
4	36770000	47

5.1.3.4 Impervious

No loss calculations were carried out on the impervious area; all precipitation on that portion of the subbasin became excess precipitation and was subjected to direct runoff (Scharffenberg and Fleming 2006). Any percentage specified was not included in computing the composite curve number. Table 5-4 shows the impervious parameter used in the loss model to calculate the losses.

Table 5-4 Impervious used in the SCS curve number loss model

Subbasin	Impervious (%)
1	9
2	12.5
3	5
4	5

5.1.3.5 Transform

Direct runoff is the process of runoff of excess precipitation on a catchment. This process refers to the "transformation" of precipitation excess into runoff. A subbasin element conceptually represents infiltration, surface runoff and subsurfaces processes interacting

together. The actual subsurface runoff calculations were performed by a transform method within the subbasin (Scharffenberg and Fleming 2006).

For our specific study, the SCS Unit Hydrograph method was used, because hydrograph data were available for each site (water level data were converted to discharge using the rating curves constructed). This method was originally developed from observed data collected from small agricultural catchments and seems appropriate for the L2 catchment. The lag is defined as the length of time between the centroid of precipitation mass and the peak flow of the resulting hydrograph. The lag times for the subbasins, as well as for the reaches (input parameter), were estimated from the hydrograph data obtained. Table 5-5 shows the lag time used in the model.

Table 5-5 Lag time for subbasins 1 to 4.

Subbasin	Lag Time (mins)
1	490
2	440
3	690
4	830

5.1.3.6 Baseflow

A subbasin element conceptually represents infiltration, surface runoff and subsurface processes interacting together. Actual subsurface runoff calculations in HEC-HMS are performed by a baseflow method contained within the subbasin (Scharffenberg and Fleming 2006). There are different methods of assigning baseflows, but the constant monthly baseflow approach was chosen. Actual data showed that baseflow levels are constant during the month. This method allows the specification of a constant baseflow for each month of the year. Monthly baseflow was estimated from water level data for each month studied and then converted to discharge by using the rating curves developed. For modelling, the

baseflow contribution for subbasin two was determined by subtracting the estimated baseflow at site 2 from that of site 1; the same was also done for subbasins 3 and 4. As shown in Table 5-6 below, the baseflow contribution reduces from winter to summer seasons. Since there is less rainfall to recharge the aquifers and groundwater abstractions are greater during summer; the pressure head of springs that contributes to baseflow of the river is reduced.

Table 5-6 Average monthly baseflow contribution observed (m³/s) for July to December, 2006

Basin	July	August	September	October	November	December
1	1.45	1.52	1.48	1.47	1.38	1.21
2	0.45	0.45	0.51	0.57	0.45	0.57
3	0.53	0.53	0.24	0.36	0.23	0.19
4	1.45	1.41	1.33	1.21	1.18	1.12

5.1.3.7 Hydrologic Routing

Routing for all the reaches (a reach is an element with one or more inflow and only one outflow) was done by using the Lag model (Table 5-7). The Lag routing model only represents the translation of flood waves. It does not include any representation of attenuation and/or diffusion processes. It is therefore best suited for short stream segments with a predictable travel time that does not vary with depth, as was seen in the L2 River. The only parameter required was the lag time in minutes. This parameter was determined by obtaining the time differences of the peak flow for each reach, as observed from the logged water level data. Inflow into the reach was delayed by the lag specified and then becomes outflow.

Table 5-7 Reach Routing

Reach	Lag Time (mins)
1	430
2	390
3	450

5.1.4 Control Specification

The control specification controls the time span of a simulation. These included a starting date and time, ending date and time as well as a time interval. A ten minute time interval was utilised, since the logger recorded data at the same time interval.

5.1.5 Calibration and Validation of Hydrologic Model

Model calibration consists of changing values of model input parameters in an attempt to match field conditions within some acceptable criteria (how well the model data fits the measured data). This requires that field conditions at a site be properly characterized. Lack of proper site characterization may result in a model that is calibrated to a set of conditions which are not representative of actual field conditions. At a minimum, model calibration should include comparisons between model-simulated conditions and field conditions.

Four parameters: curve number, initial abstraction, impervious and lag time were subject to calibration and validation within the HEC-HMS hydrologic model. The traditional Nash-Sutcliffe Coefficient of model efficiency⁵, NS, was evaluated and used as a guide to calibrate and validate the model, as well as the Coefficient of Variation, CV, which is also used for comparison of predicted and actual measurement.

⁵ The Nash-Sutcliffe Coefficient of model efficiency, NS, is a statistical criterion for evaluating hydrologic goodness of fit between measured and predicted values for each method. An NS value of 1 indicates a perfect fit between measured and predicted values for all events, whilst a value of 0 indicates that the fit is as good as using the average value of all the measured data for each event.

Calibration and validation was done over the period of August to December, 2006.

Because the water level logger installed at site 1 did not capture some of the peak flows from a few rainstorm events, this potentially affected the calibration of basin 1 within the model. To negate this problem a smaller storm (logged on 30 December) whose peak was captured by the logger was used to calibrate basin 1. The logger at site 4 was only used when the lake was calm in the calibration process, since the outflow was influenced by the lake height as well as wind (and by waves generated from the wind), therefore parameters for basin 4 were estimated based on the calibrated parameters from the other basins, taking the conditions into consideration.

Table 5-8 and Figures 5-3, 4 and 5 show the calibration and validation details for the HEC-HMS hydrological model developed (how well the model data fits the measured data). Appendix D shows a detailed summary of the calibration and validation data for the hydrologic model.

It was considered that the model performed satisfactorily for storm events of 23.2 mm and lower. A maximum Nash-Sutcliffe value of 0.77 and a minimum value of 0.51 were obtained for storm 1 (Table 5-8). A NS value of 0.69 was computed for the December storm for site 1 (one of several other storms modelled). For the storm simulated for the period 21 to 24 August, the model peaked approximately six hours after the measured data for both site 2 and 3 (Figure 5-4), but for the storm simulated for the 13 to 16 August, peak times for both sites were much closer (Figure 5-3).

Table 5-8 Calibration and Validation details for the HEC-HMS model**Storm 1**

Date 13/08/2006 - 15/08/2006
 Total precipitation 17.49 mm
 Duration 38 hours

	Site 1*		Site 2		Site 3	
	Measured	Modelled	Measured	Modelled	Measured	Modelled
	m ³ /s	m ³ /s	m ³ /s	m ³ /s	m ³ /s	m ³ /s
Mean	1.579	1.585	2.218	2.136	2.981	2.885
Standard Deviation	0.048	0.072	0.164	0.105	0.233	0.198
CV	0.031	0.045	0.074	0.049	0.078	0.069
NS		0.511		0.578		0.777

Storm 2

Date 21/08/2006 - 22/08/2006
 Total precipitation 23.2 mm
 Duration 14 hrs

	Site 1*		Site 2		Site 3	
	Measured	Modelled	Measured	Modelled	Measured	Modelled
	m ³ /s	m ³ /s	m ³ /s	m ³ /s	m ³ /s	m ³ /s
Mean	1.593	1.621	2.236	2.185	2.937	2.868
Standard Deviation	0.072	0.187	0.181	0.256	0.335	0.452
CV	0.045	0.116	0.081	0.117	0.114	0.158
NS		-2.462		0.511		0.630

Storm 3

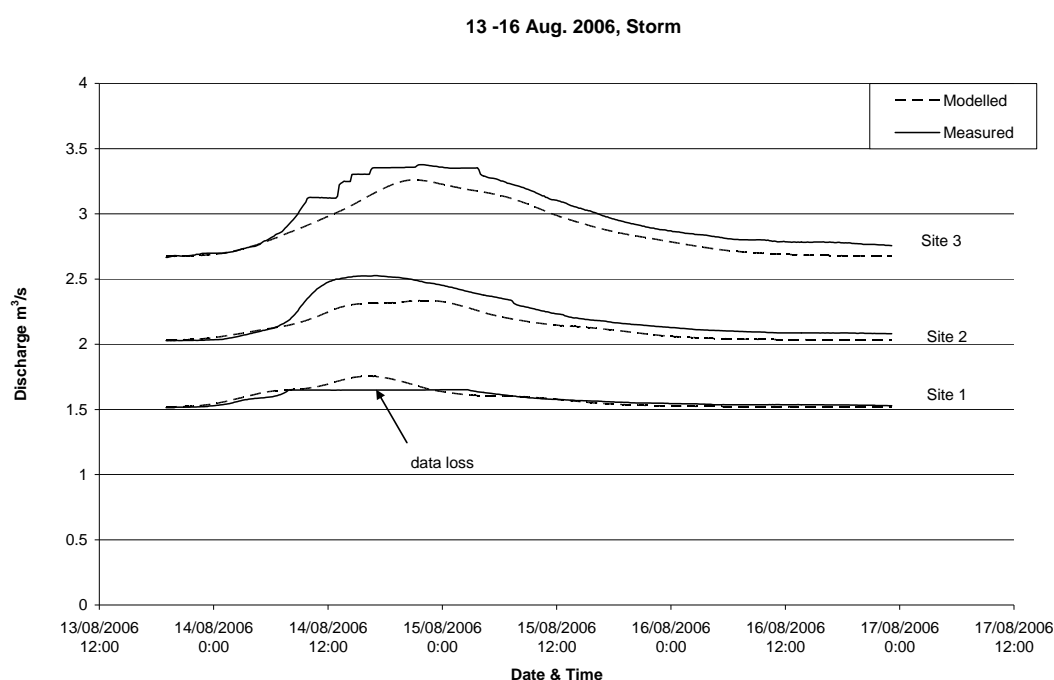
Date 03/10/2006 - 04/10/2006
 Total precipitation 74.01 mm
 Duration 24 hrs

	Site 1*		Site 2		Site 3	
	Measured	Modelled	Measured	Modelled	Measured	Modelled
	m ³ /s	m ³ /s	m ³ /s	m ³ /s	m ³ /s	m ³ /s
Mean	1.580	2.310	2.298	3.349	2.879	5.057
Standard Deviation	0.084	1.519	0.220	2.068	0.460	3.957
CV	0.053	0.658	0.096	0.618	0.160	0.782
NS		-374.571		-94.901		-81.610

* The peak of the hydrographs missing, due to shortness of logger 1 at site 1

For the storm simulated on the 03 October 2006, a total precipitation of 74.01 mm (15 year annual recurrence interval (ARI) storm), the model hydrographs were found to be 55.9 per

cent in excess of the measured hydrograph (Figure 5-5). It is believed that the reason for the difference between the measured and modelled data is due to the fact that the channel had overtopped its bank at several locations along the length of the river. Several river sections were identified where the overtopping (flooding) had occurred (confirmed by locals). These are Moirs Lane (before site 1, just out of Lincoln), a section north of McDonald Rd., Silverstream and Yarrs lagoon, just to name a few (Figure 5-6).



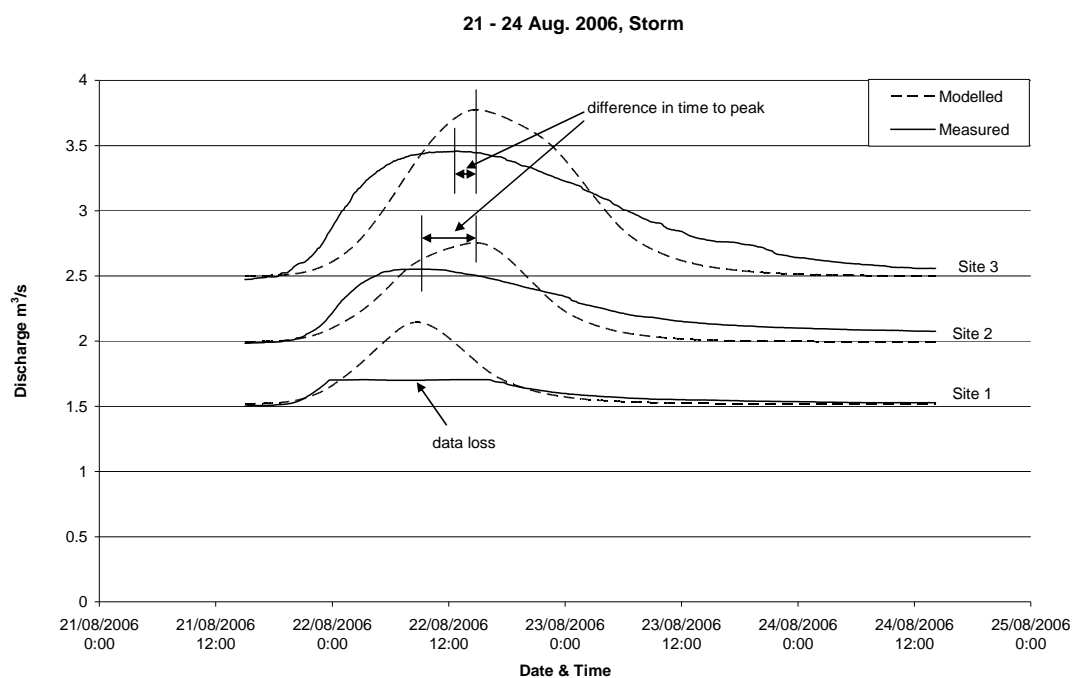


Figure 5-4 Measured vs. modelled storm event for the period of 21 to 24 August, 2006

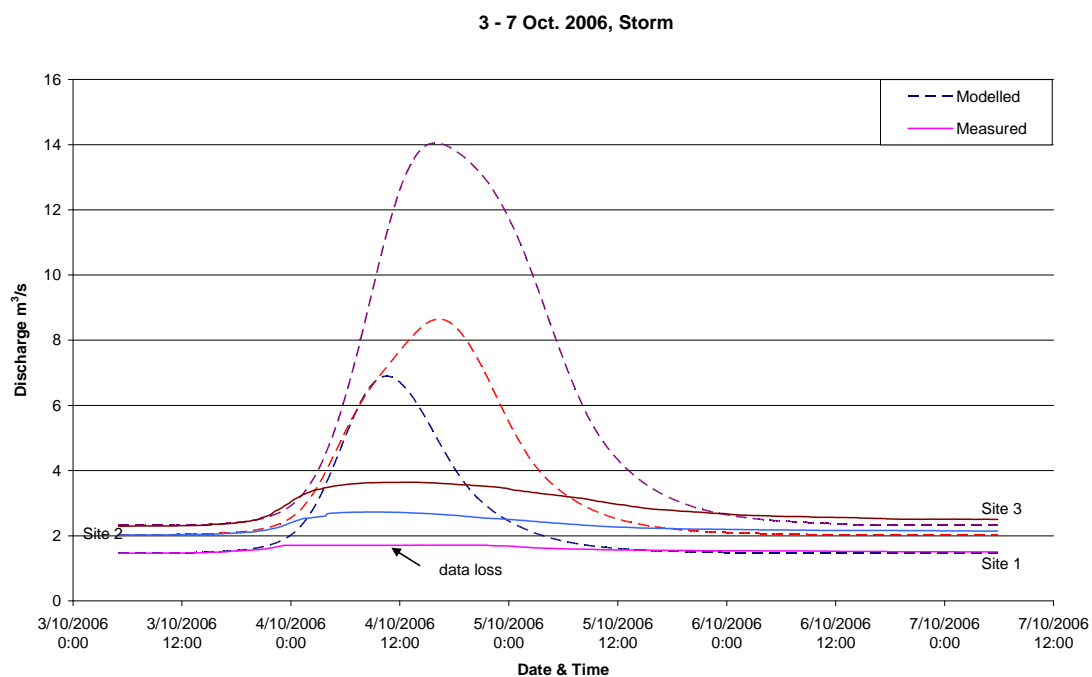


Figure 5-5 Measured vs. modelled storm events for the period of 03 to 07 October, 2006

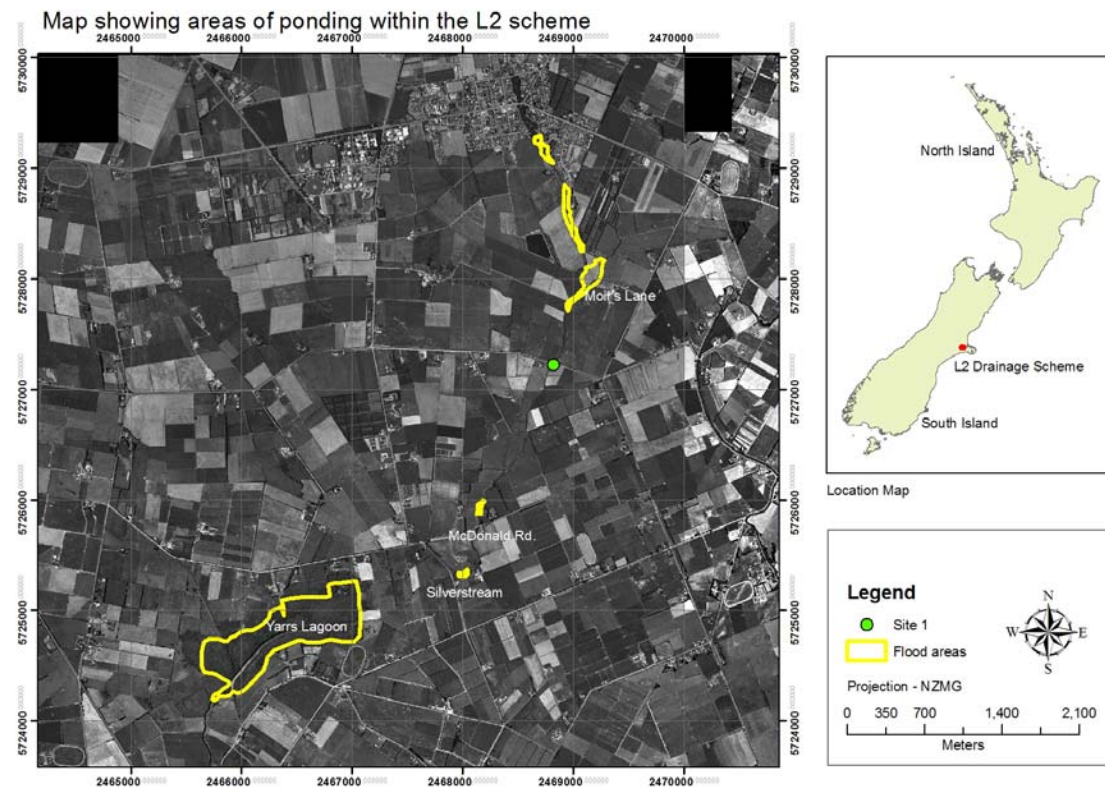


Figure 5-6 Potential flooding areas along the L2 River after an extreme rainstorm events

To support the theory that the flow had overtopped its banks during the storm that occurred on the 3 October 2006, the maximum allowable flow at each cross-section was estimated (Table 5-9) using the expression $Q = A * v$. The cross-sectional area for each surveyed cross section (between sites 1 and 4) was calculated and a maximum average channel velocity of 0.30 to 0.50 m/s was estimated (the highest recorded during streamflow measurement was 0.36 m/s), to determine this maximum allowable flow.

Table 5-9 Estimated Maximum flow (discharge) within each section using a range of velocity (0.30 to 0.50 m/s)

Cross Section #	X- Sectional Area, A m ²	Max. Estimated Discharge m ³ /s
1 (Site 1)	11.00	3.30 - 5.50
2	11.25	3.38 - 5.63
3	9.00	2.70 - 4.50
4	24.00	7.20 - 12.00
5	6.10	1.83 - 3.05
6 (Site 2)	24.00	7.20 - 12.00
7	22.70	6.81 - 11.35
8	15.40	4.62 - 7.70
9 (Site 3)	24.00	7.20 - 12.00
10	29.10	8.73 - 14.55
11	20.00	6.00 – 10.00
12 (Site 4)	31.25	9.38 - 15.63

As shown in Table 5-9, the lowest estimated discharge through the L2 River was 3.05 m³/s at cross section #5, hence the maximum discharge the channel can convey for branch 1, without overflowing its bank was 3.05 m³/s, within the study area (at Silverstream) which is close to site 2. It is important to note that other cross-sections that were not surveyed may carry less flow. Using the rating curves created to convert water level to discharge, the maximum discharge logged at site 2 was 2.73m³/s. Also taking into account that Moir's Lane is located before site 1 (out of study area) the same occurs, that is, the channel was overtopped and an area of ponding was created during high flows. Therefore, there was even less water entering the area of study than the model computed for this storm.

5.1.6 Sensitivity Analysis

A sensitivity analysis was performed, examining the impact that curve number, initial abstraction and lag time have on the hydrological model. The results from a sensitivity analysis helped to identify which input parameters should be selected with greater precision

(i.e. the ones that have a high sensitivity) and what parameters would be acceptable with a rougher approximation.

The sensitivity analysis was done by making one standard run using a storm with the calibrated parameters. The investigated parameter was then changed and a new run computed. The relative percentage difference between the used parameter values (the investigated one) was then compared with the relative percentage difference in the results. The percentage difference between the results was divided by the percentage difference in the parameter to obtain the relative percentage sensitivity. This procedure was then repeated for other parameters tested, as well as for the other storms studied.

Comparing these numbers shows which results are sensitive to changes in parameters. For this analysis the peak flow, peak time and total volume at the outlet were used as the results. Table 5-10, Figures 5-7, 8 and 9 show the result of the sensitivity analyses for each parameter analysed. The summary of the performed runs is shown in Appendix E. In the case of the curve number and the initial abstraction where each subbasin had its own value, the mean value used in the simulation was presented in tables and used for the sensitivity calculation.

Table 5-10 The storm events used

Storm	Total precipitation (mm)	Duration (hrs)
1	17.49	38
2	23.20	14
3	74.01	24

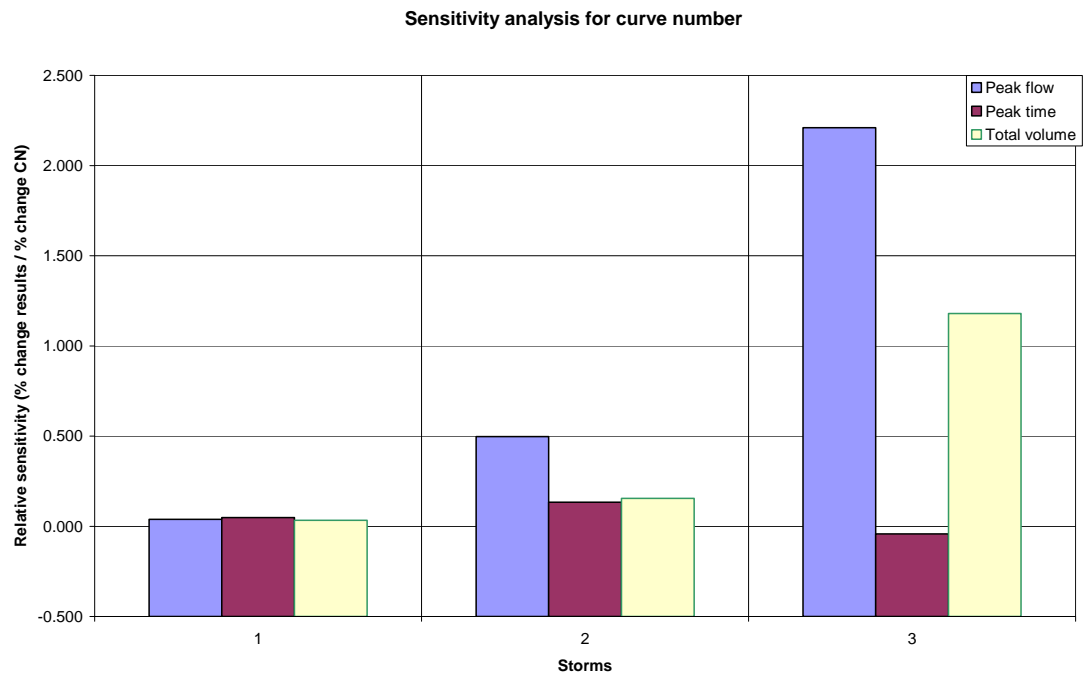


Figure 5-7 Sensitivity analysis for peak flow, peak time and total volume for curve number

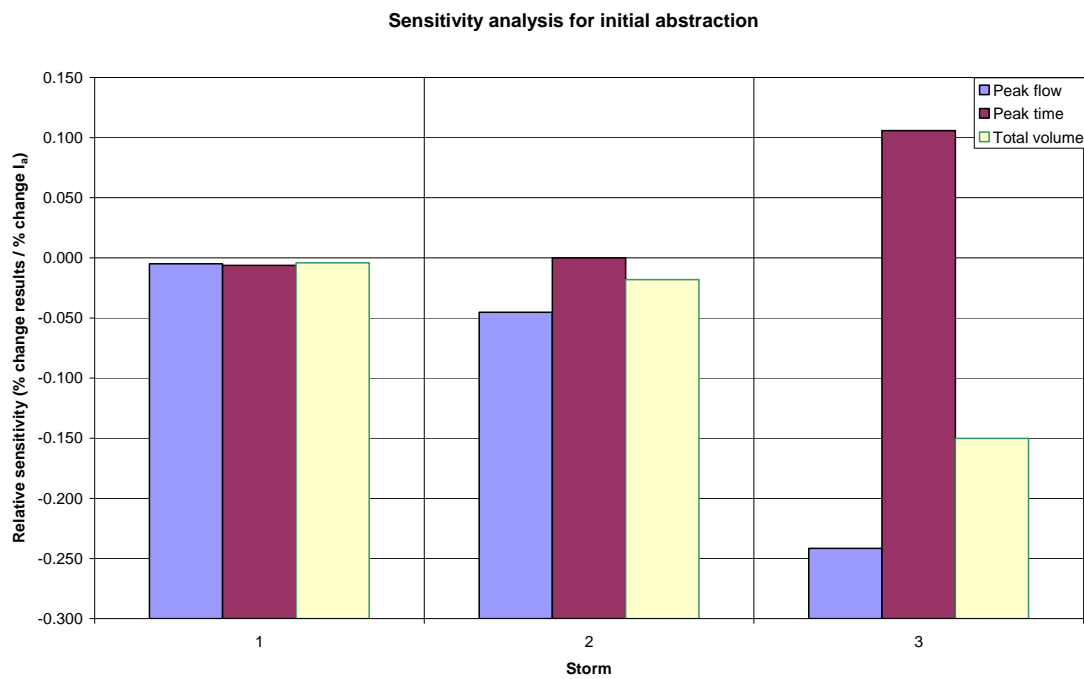


Figure 5-8 Sensitivity analysis for peak flow, peak time and total volume for initial abstraction

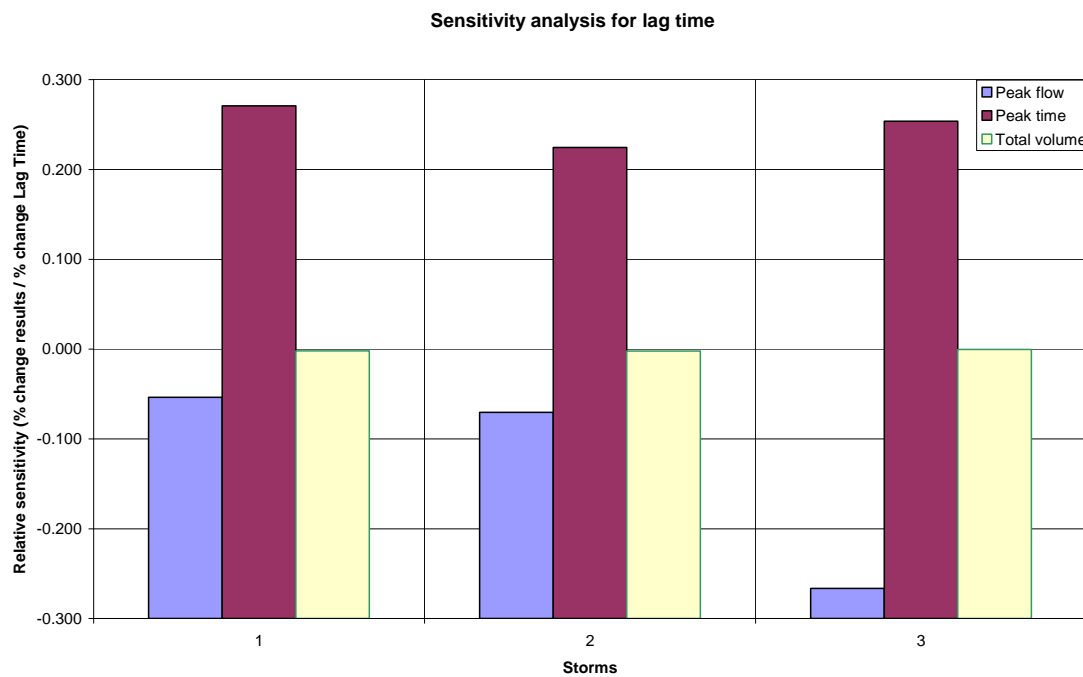


Figure 5-9 Sensitivity analysis for peak flow, peak time and total volume for lag time

A relative sensitivity value above ± 1 was regarded as a critical value, since it means that the result changes more than the parameter and therefore is highly sensitive. The sensitivity analysis shows that the highest values were for the curve number on the peak flow. The curve number was also the most sensitive parameter when it comes to total volume (Figure 5-7). These results were expected, since the curve number was used for calculating losses and since a higher curve number results in more runoff and therefore more volume and higher peaks.

The lag time was the most sensitive parameter when it comes to peak time (Figure 5-9). Lag time was the least sensitive parameter, compared to other analysed parameters (i.e. curve number and initial abstraction) for the total volume. An increase in lag time results in a lower peak from the subbasins. Hence, to transport similar volumes on a longer time frame, will result in a lower peak.

The biggest change in value for a single parameter was the initial abstraction, which was increased by more than 160 per cent (Appendix D). Naturally this seems tremendous, but when it comes to sensitivity, I_a seems insensitive, with values below 0.25 for all evaluated results.

It is important to note that as the total precipitation increases, the sensitivity of the model to the inputted parameters increases. Therefore care was taken when estimating all parameters for the model but specifically the curve number, as it is the most sensitive parameter.

5.1.7 Model Output

HEC-HMS calculates a hydrograph and time to peak for each subbasin, reach and junction hydrograph. The results of a run can be viewed as a time series for a specified subbasin, reach or junction, which can be copied and pasted into Microsoft Excel. For this study the hydrographs from the simulated scenarios, that is, for the 2, 10 and 20 year ARI storms for subbasins 1 to 3, was used as input parameters for the MIKE 11 hydraulic model.

5.2 Hydraulic Model

5.2.1 Introduction

The hydraulic model of the L2 River within the L2 catchment was constructed using MIKE 11. The MIKE 11 software package is a versatile and modular engineering tool for modelling conditions in rivers, lakes/reservoirs, irrigation canals and other inland water systems. It is able to model relatively low, shallow flows, which were analogous to the flow seen in the L2 River. Other modelling packages, such as HEC-RAS, which are specifically for river modelling, are more focused towards large steady flows. Another reason for

selecting MIKE 11 was the availability of a software licence in the Department of Civil Engineering. As Dark (2005) claims,

Computational modelling allows water levels and discharges at any point along the modelled reach to be investigated, whereas field observations and analysis of hydrological data are limited by the number of measurement points (Dark 2005).

Mosley (1992) also stated that,

Models are not scientific theories, though the two are closely linked. A model provides a means for exploring a theory and for testing hypotheses (Mosley 1992).

The aim of the computational modelling programme was to quantify what the effects of higher lake levels coupled with large storms events (2, 10 and 20) will be on the L2 Drainage scheme.

5.2.2 Model Setup

The MIKE 11 user interface comprises a number of different editors in which data can be implemented and edited independently of each other. The integration and exchange of information between each of the data editors was achieved by the use of the MIKE 11 Simulation editor. For this study the network, cross-section, boundary and hydrodynamic modules were used. The hydrodynamic module is the nucleus of the Mike 11 modelling systems and forms the basis for most modules.

5.2.3 Network Module

The river network editor gives an overview of the current setup and provides a common link to the various MIKE 11 editors (DHI Water and Environment 2004). This editor is the central unit of the MIKE 11 graphical user interface and consists of two views, a graphical view, where graphical editing of the network was performed and a tabular view, where the

river network data were presented in tables. The L2 River was defined and connected to form three branches: branch 1; a length of 2314 m (site 1 to Englishs Road bridge), branch 2; a length of 3344 m (Englishs Road bridge to Pannetts Rd. bridge) and branch 3; a length of 3487 m (Pannetts Road Bridge to site 4). The graphical view of the Network is shown in Figure 5-10.



Figure 5-10 Network editor showing the graphical view of the L2 River (Land Information New Zealand 1:50000 Topographic Map sheets)

5.2.4 Cross-Section Module

The cross-section editor was used to define the cross-sections of selected locations along the L2 River and consists of two data sets, the raw (Figure 5-11) and processed data. The raw

data represents the physical shape of the cross-section using (x, z) co-ordinates. The processed data were calculated from the raw data and contain corresponding values for level, cross-sectional area, flow width and hydraulic/resistance radius. After the network has been defined, the twelve cross-sections, along with the chainage and respective datum (elevation) were then entered at locations (where cross-sections were executed) to define the channels.

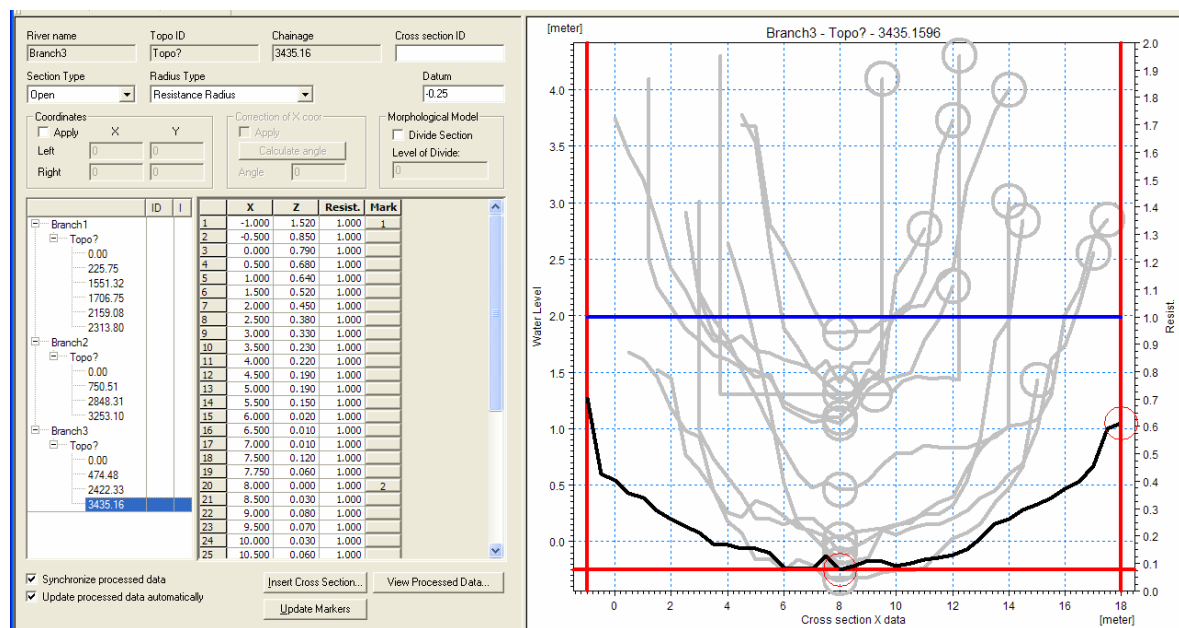


Figure 5-11 Cross-section showing raw data and cross-sections of the L2 River

5.2.5 Boundary Module

The boundary editor was used to specify boundary conditions to the MIKE11 model. It was used not only to specify common boundary conditions such as water levels and inflow hydrographs but also for the specification of lateral flows along river reaches. The boundary editors consist of the time series editor and the boundary editor. The time series data dialog consists of two views, a tabular and a graphical view. The boundary editor dialog was used to specify the boundary conditions for the hydrodynamic calculation. The MIKE 11 model required water level or discharge data boundary events to be specified at all boundary nodes

in the model, that is, discharge (obtained from converting the water level data to discharge using the rating curve created) at site 1 and water level at site 4. The boundary data for an event were entered as a time series, with a ten-minute time step. Screen shots of the time series editor and the boundary editor are shown in Figure 5-12 and Figure 5-13.

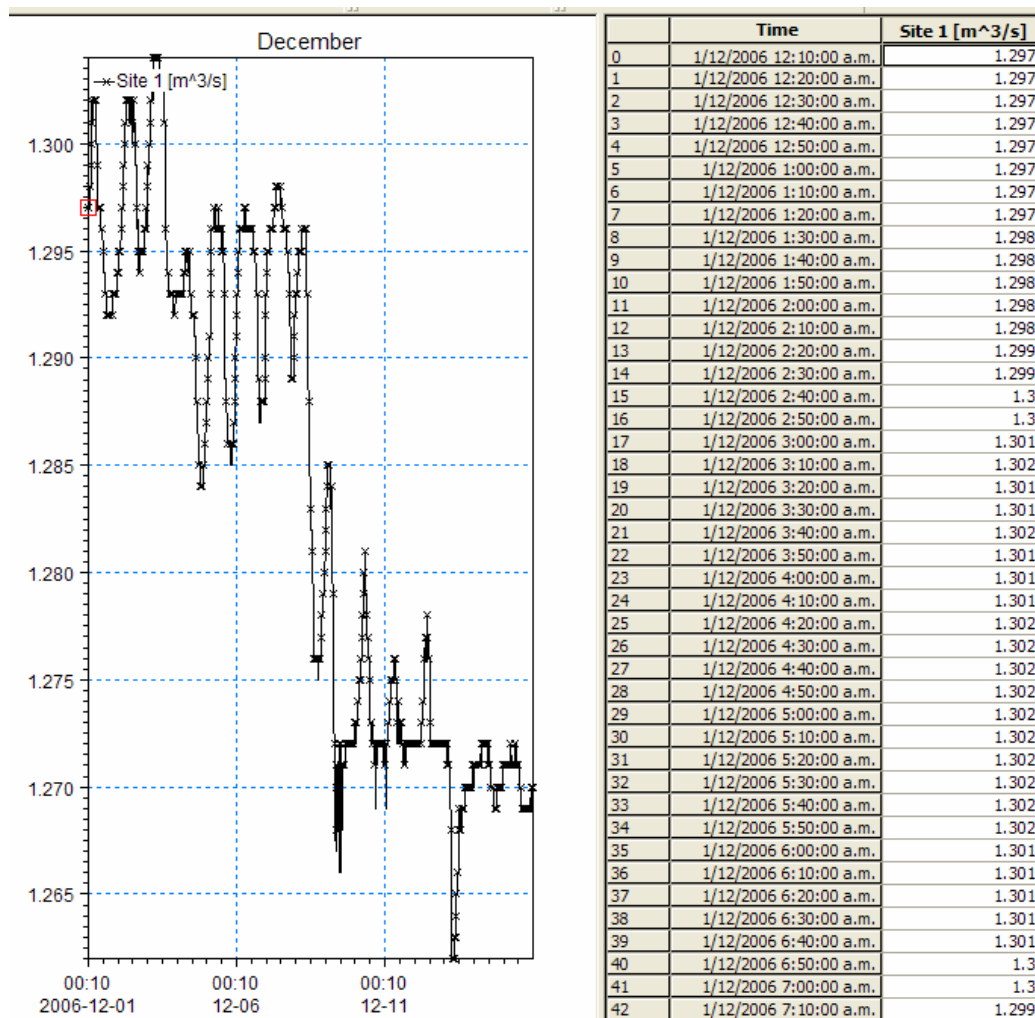


Figure 5-12 MIKE 11 time series editor showing graphical and tabular flow rate data for site 1

	Boundary Description	Boundary Type	Branch Name	Chainage	Chainage	Gate ID	Boundary ID
1	Open	Inflow	Branch1		0	0	
2	Open	Water Level	Branch3	3435.159625394		0	
3	Point Source	Inflow	Branch2		0	0	
4	Point Source	Inflow	Branch3		0	0	

☒ Include HD calculation
☐ Include AD boundaries
☐ Mike 12

	Data Type	TS Type	File / Value	TS Info
1	Discharge:	[TS File]	L2_Site 1_Dec_TS1.d ... Edit	Site 1

Figure 5-13 MIKE 11 boundary editor

The boundary at the upstream end of a branch, that is, the “inflow” boundary condition (since water is entering the channel) was entered in the times series editor and then selected in the boundary editor. The boundary data at the downstream end, that is, the water level data, were entered into the time series dialog and then selected in the boundary editor. The inflow from the drains (basin) between sites 1 and 4 was included in the model as lateral inflow (point source) at sites 2 and 3, since this represents the actual flow condition seen within the catchment. If this was not included in the model building process, the water level within the channel when simulated, would be lower than it should have been. As a result of this, the Manning’s n roughness coefficient would be over-estimated, since the higher the roughness coefficient, the higher the same quantity of water would be in the channel.

5.2.6 Hydrodynamic Module

The hydrodynamic editor allows the user to define values for a number of variables used during the hydrodynamic computation (DHI Water and Environment 2004). The initial conditions were set as shown in Figure 5-14. The parameter of main concern in this editor was the Manning’s n (Table 5-11), since this parameter affects the water level in the channel. The Manning’s n values used for December were calculated, using Equation 2-5, with the

parameters obtained from fieldwork. These values together with additional field work were then used to estimate Manning's n value (Table 5-11) for other months. As can be seen, the Manning's n value increased during the period of study, coinciding with the actual field condition, that is, increase in weed growth from winter to summer.

Figure 5-14 Hydrodynamic parameters

Table 5-11 Calibrated Manning's n Values

Calibrated Monthly Manning's n values

Months	Manning's n Values		
	Branch 1	Branch 2	Branch 3
July	0.073	0.067	0.055
August	0.075	0.070	0.058
September	0.070	0.082	0.062
October	0.085	0.093	0.065
November	0.092	0.103	0.071
December	0.099	0.114	0.077

5.2.7 Model Settings

The main computational parameter that is set by the user in MIKE 11 is the time step. In general, complicated modelling situations required shorter time steps for the simulation to be numerically stable. For a given simulation a shorter time step will produce more accurate results, but will require more time to compute. The time step should be adequate to provide an accurate representation of a wave progressing down a river. If there are structures located along the length of the river, a smaller time step is required. For this study, a ten-minute time step was used for the simulations.

5.2.8 Model Output

MIKE 11 calculates water level and discharge at each cross-section for each time step specified. The results of the simulations were viewed in MIKE View as animations, as a time series for a specified cross-section or along the length of the river. Results were also exported to Microsoft Excel for further analysis. MIKE View also computed the flood depth for all three branches of the L2 River. The most useful output was the water level time series at the various cross-sections along the length of the L2 River.

5.2.9 Calibration and Validation of Hydraulic Model

The MIKE 11 hydraulic model was calibrated and validated by modifying Manning's n values according to field observation. The percentage deviation between measured and modelled data was evaluated and used as a guide to calibrate and validate the model. The Coefficient of Variation, CV, was also used for comparison of modelled and actual measurements.

Calibration and validation was done over the period of July to December, 2006, on a monthly basis. Four simulation periods which represent the variation of flow and weed

found within the channel are presented. These are periods in July – when the weeds are very low, December – when the weeds are high and dense and two large storms in August. Since the logger installed at site 1 did not capture some of the peak flows from rainstorm events, this affected the calibration of the model at site 2 and site 3 for a storm event (less inflow in the channel) to some extent.

To overcome this problem, the hydrograph for the storm measured on the 21 to 24 August 2006 for site 2 was used as the “inflow” condition. Therefore, no lateral inflow was entered in the time series and boundary editor for site 2. The difference in the measured hydrographs between sites 2 and 3 was then used as the lateral inflow for site 3 in the simulation. As a result of this, the measured water level at both sites 2 and 3 should closely match that of the model, since the measured water level (converted to discharge) was used as the inflow for this simulation (Table 5-12 and Figure 5-16). Also, the rainstorm event that occurred for the same period, that is, 21 to 24 August, 2006 was simulated using the HEC-HMS model and the resulting hydrographs for subbasins 1, 2 and 3 were used as “inflow” parameters for the hydraulic simulation. The hydrograph of subbasin 4 was not used, since site 4 was fixed, that is, the water level data were entered as a time series and were selected in the boundary editor.

Table 5-12 and Figures 5-15, 16, 17 and 18 show the calibration and validation details for the MIKE 11 hydraulic model developed (how well the model data fits the measured data).

Table 5-12 Calibration and validation

Period 25th - 31st July, 2006

	Site 2		Site 3	
	Measured	Modelled	Measured	Modelled
Mean	2.143	2.153	1.405	1.405
Standard Deviation	0.009	0.008	0.005	0.013
Coeff. Of Variation	0.004	0.004	0.003	0.009
Max	2.171	2.173	1.414	1.579
Min	2.029	2.141	1.390	1.390
Percent Deviation		0.454		0.466

Period 21st - 24th August, 2006

Using site 2 measured hydrograph as an "inflow" parameter

	Site 2		Site 3	
	Measured	Modelled	Measured	Modelled
Mean	2.381	2.354	1.683	1.597
Standard Deviation	0.125	0.101	0.182	0.156
Coeff. Of Variation	0.052	0.043	0.108	0.098
Max	2.598	2.520	1.964	1.844
Min	2.206	2.208	1.431	1.352
Percent Deviation		1.102		4.884

Period 21st - 24th August, 2006

Using results from the HEC simulation as input parameters

	Site 2		Site 3	
	Measured	Modelled	Measured	Modelled
Mean	2.381	2.283	1.683	1.522
Standard Deviation	0.125	0.071	0.182	0.090
Coeff. Of Variation	0.052	0.031	0.108	0.059
Max	2.598	2.434	1.964	1.696
Min	2.206	2.060	1.431	1.419
Percent Deviation		3.974		9.476

Period 1st - 15th December, 2006

	Site 2		Site 3	
	Measured	Modelled	Measured	Modelled
Mean	2.408	2.390	1.438	1.399
Standard Deviation	0.002	0.001	0.009	0.006
Coeff. Of Variation	0.001	0.000	0.007	0.004
Max	2.415	2.392	1.459	1.420
Min	2.399	2.389	1.424	1.392
Percent Deviation		0.739		2.720

For the simulations done when the weed was low within the channel (25 to 31 July) and when the weed was very dense (1 to 15 December) with no rainstorm event, the model performed exceptionally well, with the maximum percentage deviation 2.70 and a minimum value of 0.45. For the simulation executed using the measured hydrograph of site 2 as the inflow parameter, the model also performed well (Figure 5-16) with a maximum coefficient of variation of 0.098 and a minimum of 0.043. For the storm modelled using HEC-HMS for the period 21 to 24 of August 2006, the average maximum water level at sites 2 and 3 was found to be 11.27 per cent below the measured water level (Figure 5-14).

Based on the data presented in Table 5-14, it was considered that the model performed satisfactorily; with a maximum percentage deviation value of 9.48 and a minimum value of 0.45. Since the model performed well for the other modelled periods, it is believed that the rating curve constructed for each site, which was adversely affected by the growth of weed in the channel (raising the water level) has affected the calibration of both models to some degree as seen by the results presented in Tables 5-8 and 11. Figures 5-15, 16, 17 and 18 also show calibration and validation results from the hydraulic model for all four simulation periods.

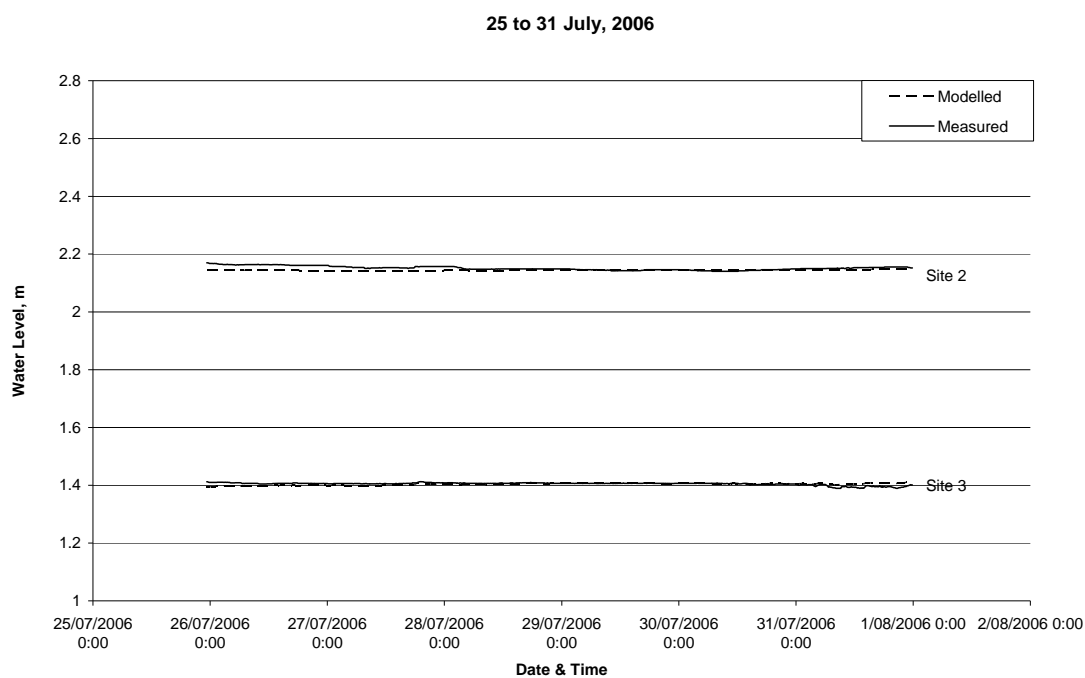


Figure 5-15 Comparison of measured vs. modelled for the period of 25 to 31 July for sites 2 and 3

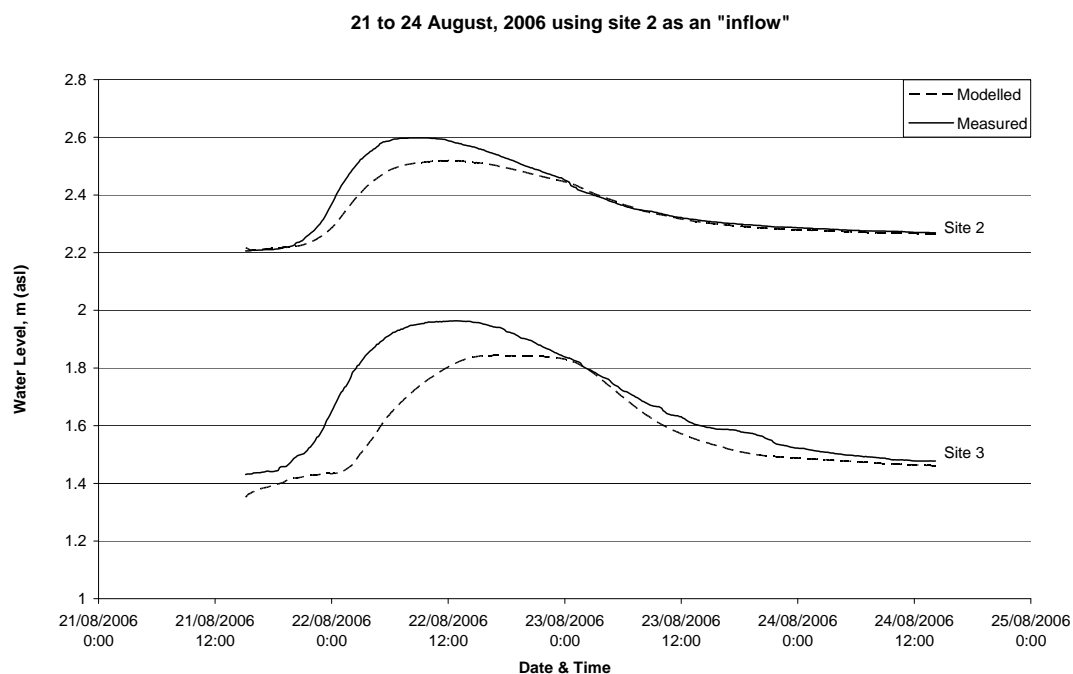


Figure 5-16 Comparison of measured vs. modelled for the period of 21 to 24 August using site 2 as an inflow parameter for sites 2 and 3

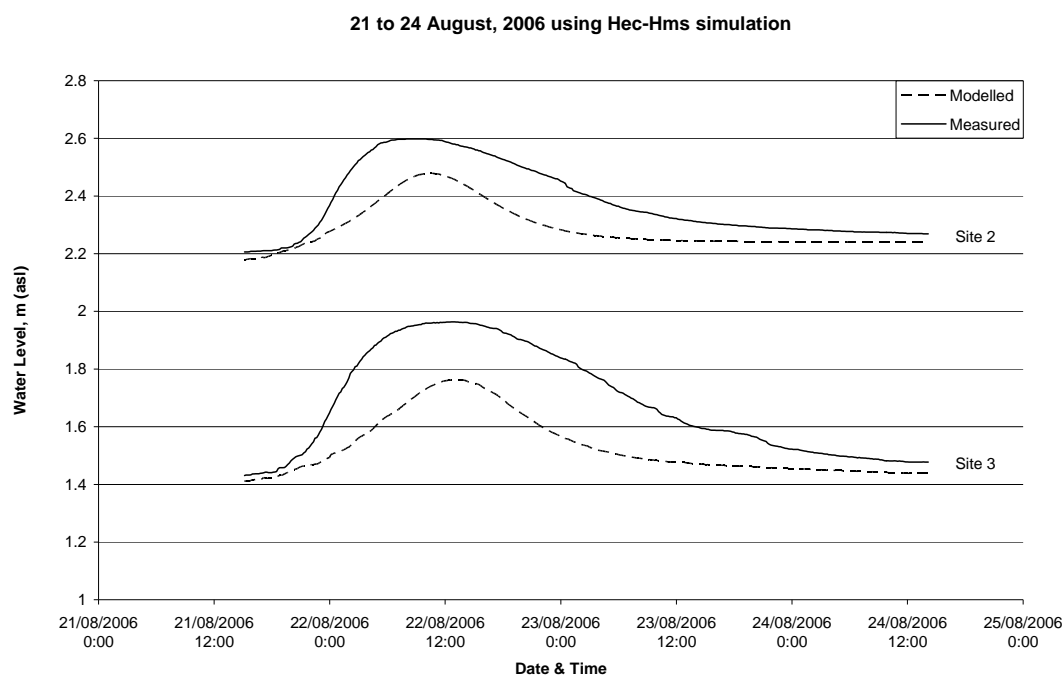


Figure 5-17 Comparison of measured vs. modelled for the period of 21-24 August using the HEC simulation as inflow parameter for sites 2 and 3

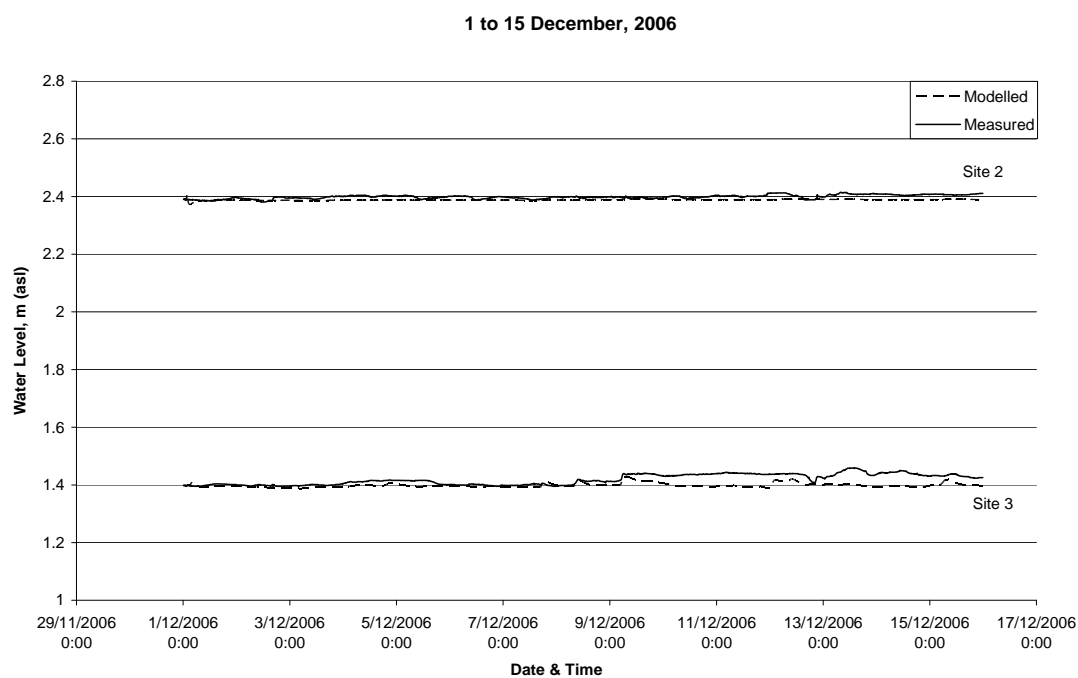


Figure 5-18 Comparison of measured vs. modelled for the period of 1-15 December for sites 2 and 3

5.2.10 Sensitivity Analysis

A sensitivity analysis was performed, looking at the impact that Manning's n , slope and the cross-section has on the results. The results from the sensitivity analysis helped in identifying which input parameters should be selected with greater precision (i.e. the ones that have a high sensitivity) and what parameters will be acceptable with a rougher approximation.

The sensitivity analysis was completed by making one standard simulation for a period, with the calibrated parameters. The investigated parameter was then changed and a new simulation computed. The relative percentage difference between the used parameter (the investigated one) values was then compared with the relative percentage difference in the results. The percentage difference between the results was divided by the percentage difference in the parameter to obtain the relative percentage sensitivity. This procedure was then repeated for other parameters tested. For this analysis the water level was used as the results.

5.2.10.1 Model sensitivity to slope and cross-section

The elevation of cross-section 6 was decreased from 1.099 m to 0.9099 m asl (hence increasing the slope of reach 5 from -0.0003 to 0.0009 (Table 4-5)) to determine the sensitivity of the slope parameter. Also, the sensitivity of the model was determined as to discover how many points were required to define cross-sections within the model. The sensitivity for both slope and cross-section was only conducted for the period 25 to 31 July 2006. Table 5-13 and Figure 5-19 show a summary over the performed runs and the result of the sensitivity analyses for Manning's n ; each branch had its own value, but the mean value was used for the calculation.

Table 5-13 Sensitivity Analysis Summary

Period 25th - 31st July, 2006						
	Original			Analysed		
	Branch 1	Branch 2	Branch 3	Branch 1	Branch 2	Branch 3
Manning's n	0.073	0.067	0.055	0.100	0.100	0.100
Average Manning's n			0.065			0.100
% difference to original value						
Model Results						
Average max. water level (site 2 and 3) (m)				1.779		
				2.054		
% difference to original value						
Relative sensitivity (% change results / % change Manning's n)						
Water Level						

Period 21st - 24th August, 2006						
	Original			Analysed		
	Branch 1	Branch 2	Branch 3	Branch 1	Branch 2	Branch 3
Manning's n	0.080	0.077	0.061	0.100	0.100	0.100
Average Manning's n			0.073			0.100
% difference to original value						
Model Results						
Average max. water level (site 2 and 3) (m)				2.032		
				2.313		
% difference to original value						
Relative sensitivity (% change results / % change Manning's n)						
Water Level						

Period 1st - 15th December, 2006						
	Original			Analysed		
	Branch 1	Branch 2	Branch 3	Branch 1	Branch 2	Branch 3
Manning's n	0.099	0.114	0.077	0.130	0.130	0.130
Average Manning's n			0.097			0.130
% difference to original value						
Model Results						
Average max. water level (site 2 and 3) (m)				1.923		
				2.119		
% difference to original value						
Relative sensitivity (% change results / % change Manning's n)						
Water Level						

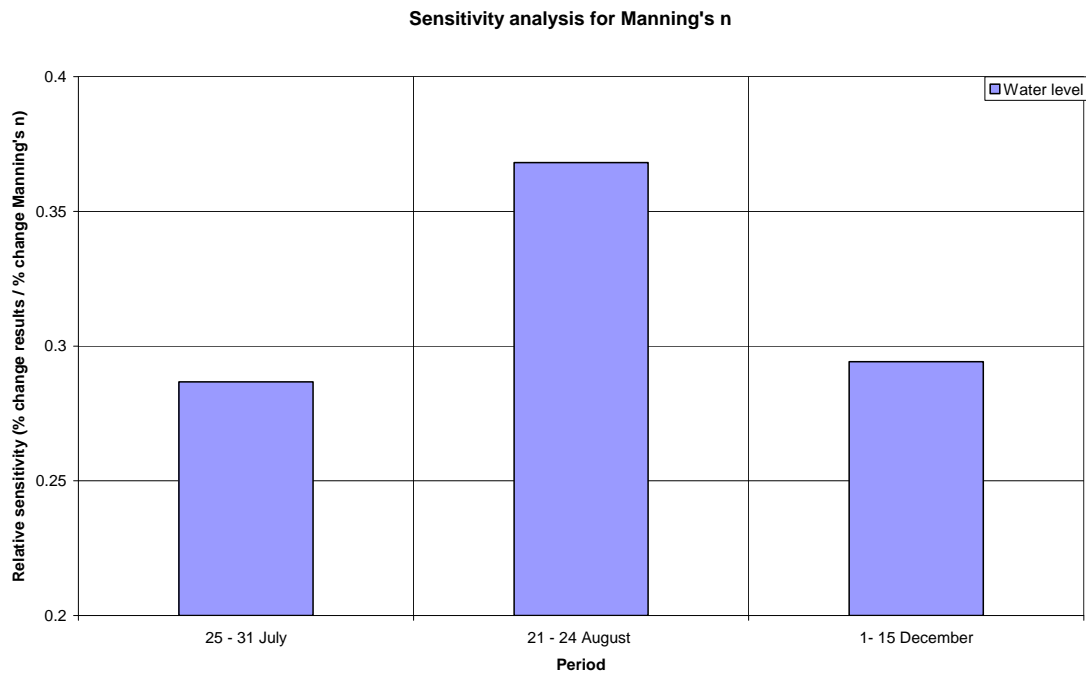


Figure 5-19 Sensitivity analysis for water level for Manning's n

Relative sensitivity values above ± 1 were regarded as critical values; since this represented that the model output changes more than the tested parameter and therefore was highly sensitive. Manning's n was a sensitive parameter, but not a critical one, with a maximum sensitivity value of 0.37 and a minimum of 0.29. The larger the Manning's n value used in the model, the higher the water level. An increase of Manning's n (average) by 42.06 per cent resulted in the average maximum water level rise of 13.17 per cent at sites 2 and 3.

A decrease in elevation at cross-section 6 by 17.21 per cent (an increase in reach 5 slope by 366.67 per cent) has resulted in a decrease in the maximum water level at cross sections 3, 4, 5 and 6, a distance of over 762 m (Figure 5-20), with cross-section 5 affected the most (7.00 per cent).

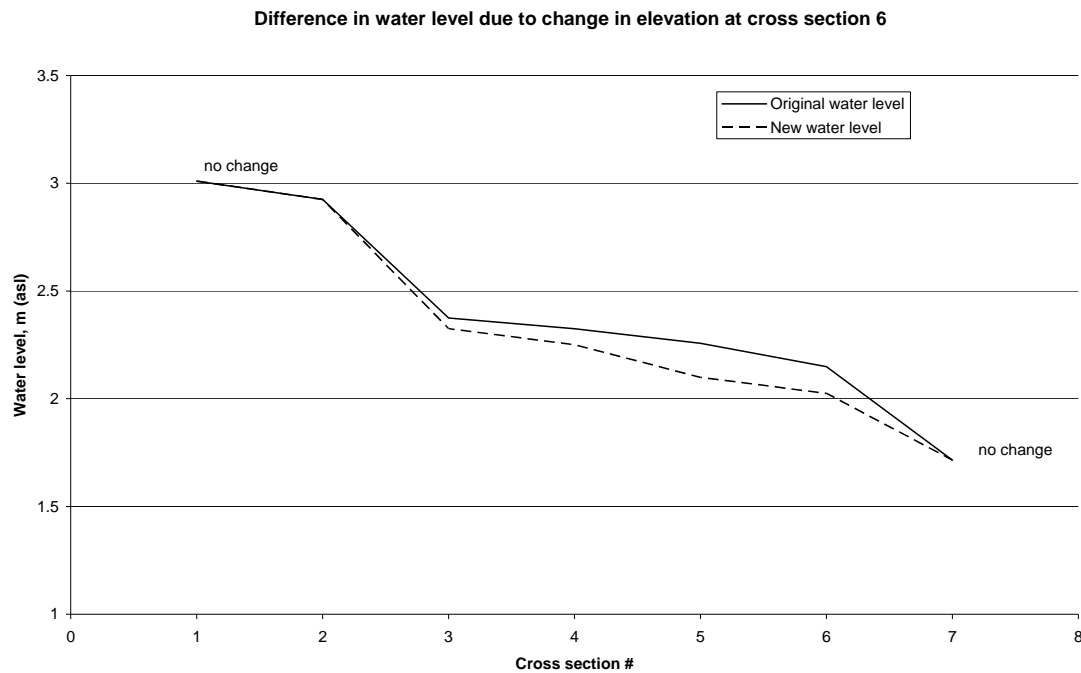


Figure 5-20 Change in elevation at cross-section 6

For the sensitivity analysis of the cross-sections, the cross-sections within the model were defined at 1.5 m and 2.5 m intervals instead (fewer surveyed points) of the 0.5 m (i.e. as actually measured in the field). Figure 5-21 shows cross-sections 1 and 12 as defined within the hydraulic model using surveyed points at 0.5, 1.5 and 2.5 m intervals. As shown in Figure 5-21, the fewer the surveyed points for cross-section 1, the elevation and hence the slope of the channel bottom was affected (different from when using the 0.5 m intervals). Whereas, with cross-section 12, the elevation or slope of the channel bottom was least affected.

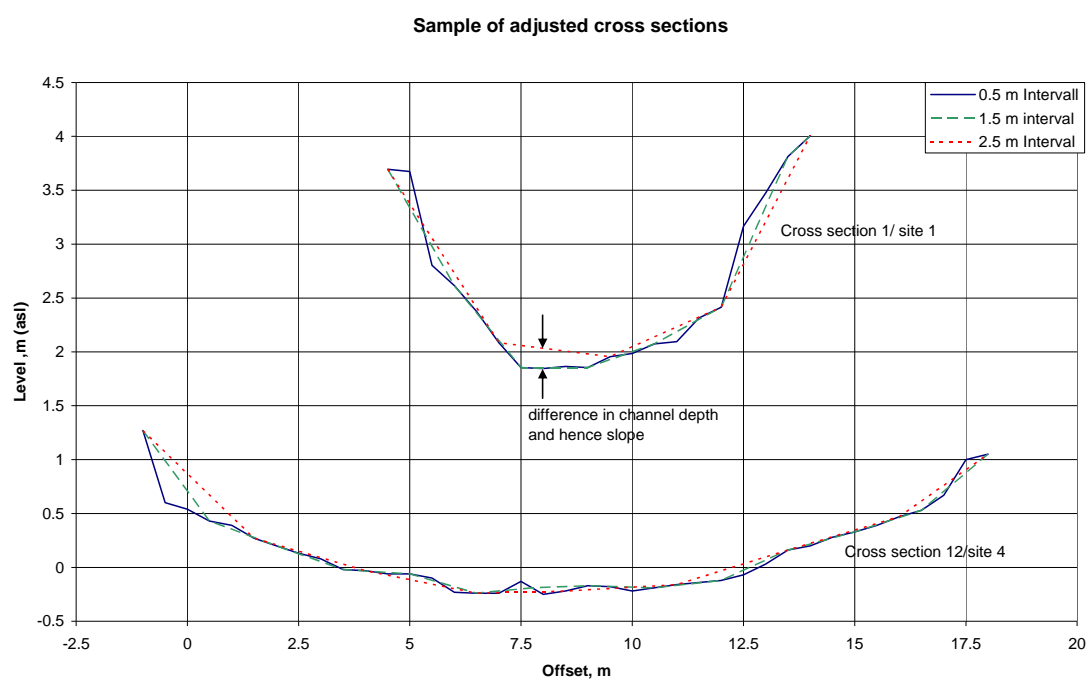


Figure 5-21 Samples of cross sections defined at 0.5, 1.0 and 2.5 m intervals used in the sensitivity analysis

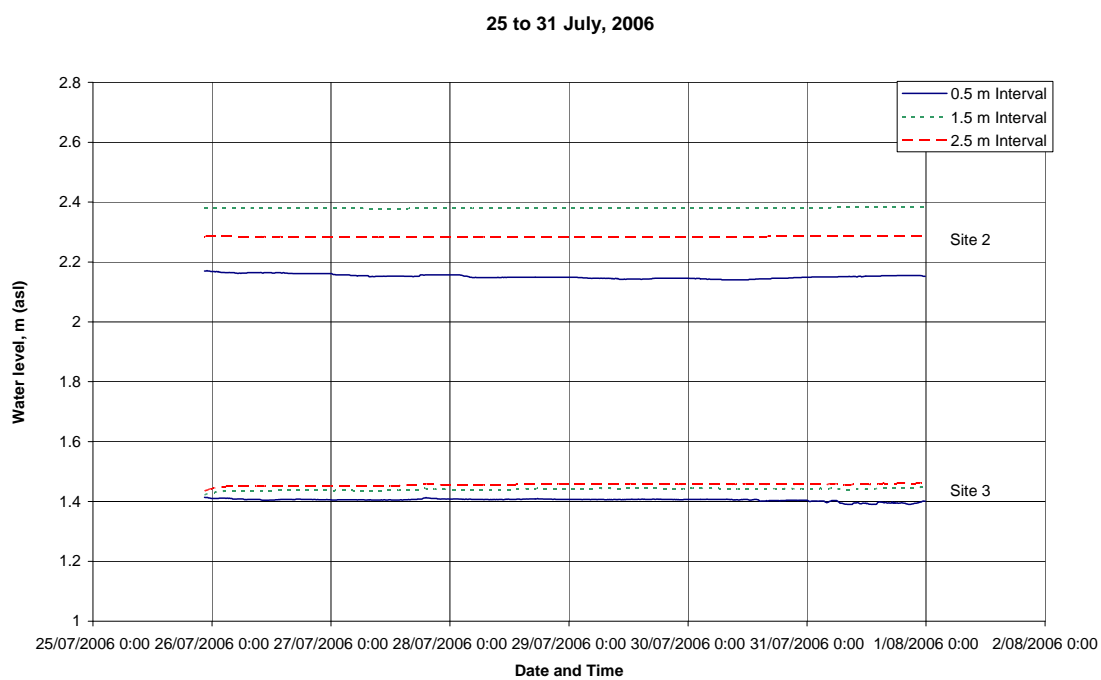


Figure 5-22 Resulting water levels after defining all cross-sections at 0.5, 1.5 and 2.5 m intervals

Site 2 was found to be affected the most. Because the upper reach of the L2 River is narrower than the lower reach, a less defined survey (as in this case 1.5 or 2.5 m interval) would misrepresent the profile of the cross-section. Since the least defined cross-section affects the elevation and the slope of the channel, this results in an increase in the water level within the channel, in this case by 10.75 per cent (1.5 m interval) (Figure 5-22). Likewise, since the lower reach is wider, fewer points can be surveyed. It was found that this does not affect the water level; a mere 3.69 per cent increase in water level was found (Figure 5-22).

6 Discussion

6.1 Efficiency of the L2 Drainage System

The Selwyn District Council (SDC) has the overall responsibility for the maintenance and management of the L2 River as well as the extensive system of drains within the L2 catchment. The L2 drainage district has a total length of 64.4 km of classified drains that is maintained by the SDC (Iremonger 2005). In addition to this there is an unknown length of private drains, which services individual property needs and is maintained by the individual property owners. It is likely most, if not all, of these private drains empty into the managed system.

The L2 River and the drainage network are currently cleaned on a yearly basis. The weed cutting programme is undertaken by the L2 Drainage Committee of the Selwyn District Council. The optimum time for cleaning drains is just prior to any high rainfall period as this reduces the time for weeds to re-establish. Aquatic plants become a problem when species introduced to New Zealand are present and conditions (such as adequate light, temperature and nutrients) are right for them to grow rapidly.

When the above-mentioned conditions are suitable, submerged weeds will form mounds beneath the water, sometimes stretching from bank to bank and forming a large mass that channels the water into a fast flowing current around the weed masses (Figure 6-1). When these conditions continue to prevail, the weeds gradually grow up towards the water surface forming a dense, virtually impenetrable, mass on the bottom and a less dense mass actively growing near the surface (Figure 6-2).



Figure 6-1 Weeds within the channel looking downstream of Pannetts Road bridge



Figure 6-2 Weeds growing on the water surface, upstream of cross-section 9

High weed content within the L2 channel has two possible effects: an environmental one on other plants and animals within the stream, and a physical effect. The major physical effect is an increase in the water levels associated with the weed growth, as was observed by the water level loggers (Figure 6-3). As shown, sites 1, 2 and 3 all showed a gradual increase in water level (1 to 19 December), even though there was no rainstorm for the corresponding period and as shown in Figure 6-3, lake level was constant (except for wind effect).

A partial clearing of the weeds (by the use of a hydraulic digger) within the channel between McDonald Road and Englishs Road, that is, a section downstream of site 1 and upstream of site 2, cleared late in December 2006, has resulted in a reduction of the water level of 0.25 m at site 1 and 0.2 m at site 2. It is important to notice that this occurred even though site 4 water levels were constant (was not opened to the sea) (Figures 6-3 and 4).

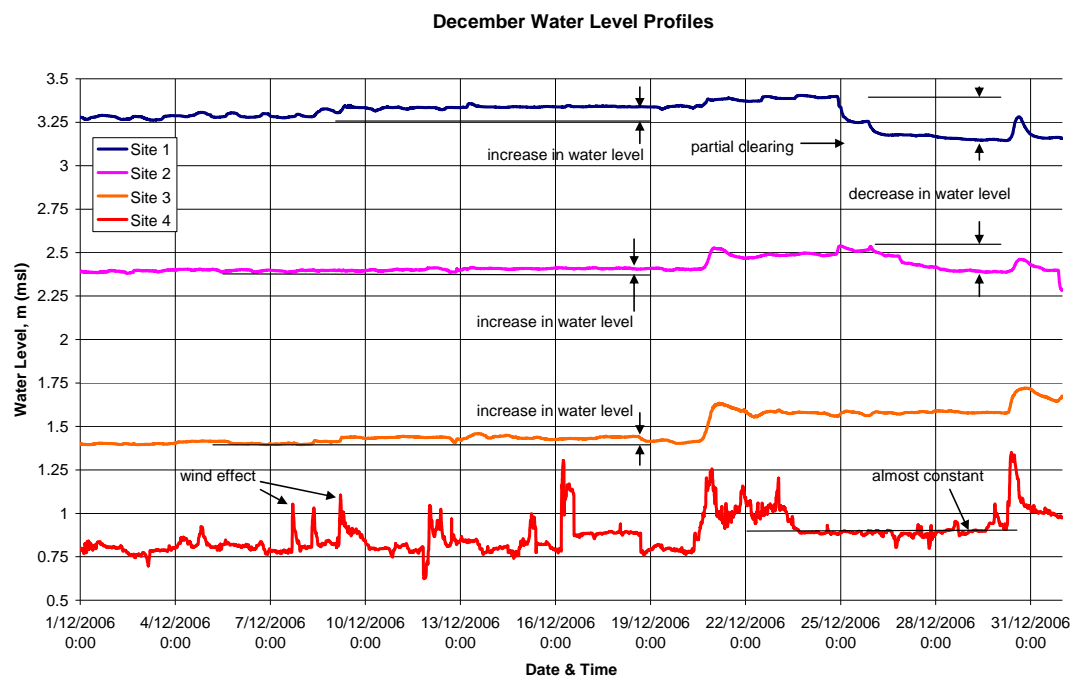


Figure 6-3 Drop in water levels after partial weed clearing done in December



Figure 6-4 De-weeding of the L2 River between McDonald Road and Englishs Road

Studies around the world, and specifically one conducted in England (Haslam 1978), have indicated that streams under 20 m wide and less than 2 m deep are likely to have ponding or flooding problems associated with plant growth. This is also the case for the L2 River. The plants affect the ability of the channel to convey water in two ways. Firstly, all aquatic plants offer some resistance to flow with the amount depending on the plants' size, structure and shape. The second effect arises as the plants get denser and water moves more slowly through them (a reduction of the water velocity, as the weed grows (Figure 4-11)) and hence the channel cross-sectional area is effectively reduced.

The aquatic weeds (submerged plants) seen in the L2 River were mainly Canadian pond weed (*Elodea canadensis*) and curly pondweed (*Potamogeton crispus*), while monkey musk (*Mimulus guttatus*), watercress (*Rorippa nasturtium-aquaticum*) (also seen growing submerged) and floating sweetgrass (*Glyceria fluitans*) were found to be semi-emergent

plants. The semi-emergent plants extended from the banks along the channel and out into the waterway of the river, channelling the water around them and trapping debris. They also caused an increase in water level as their stalks became clogged with debris and silt. However, since semi-emergent plants grow from the bank and leave the faster flowing central channel free of blockage, a greater volume of semi-emergent plants will be required to give the same effect of flow obstruction as weeds that were submerged in the central channel. As the water level continued to rise because of the growth in weeds, sections of the river were either overtopped or very close to being overtopped (Figures 6-5 and 6).

Since water level rise within the L2 channel has been associated with the weed growth the height of the banks will determine whether the water level rise caused by the weeds is sufficient to cause flooding of surrounding land. There are a number of sections where the rise in water level associated with the high weed growth will cause ponding/flooding. These locations include the right bank of cross-section 3 located north of McDonald bridge, the left bank of cross section 5 at Silverstream and the left bank of Yarrs lagoon (i.e. between cross-sections 7 and 8) (Figures 5-12, 6-5 and 6).



Figure 6-5 Cross-section (7) with water level close to overtopping the channel left bank.

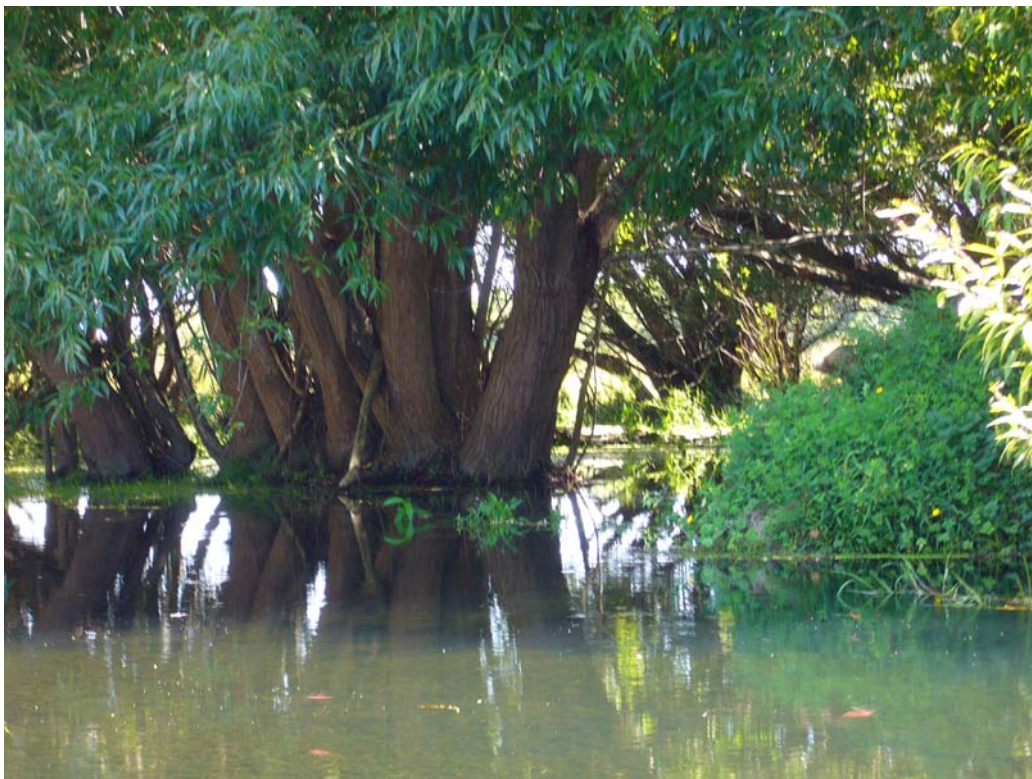


Figure 6-6 Flooding in Yarrs Lagoon as a result of high water level associated with continued weed growth (January 2007).

The high water levels as a result of the weed growth are not the only potential cause of flooding in the L2 River. High water levels in Lake Ellesmere will also cause flooding to adjacent farmlands. However, if there is a large amount of weed within the L2 channel, flooding caused by high lake levels may be exacerbated by the weeds, coupled with a major rainstorm event, in addition to a strong south-easterly wind. Lake Ellesmere and the L2 drainage scheme is no stranger to floods, with three major floods occurring within the last twenty years (1986, 1992 and 1994 (See Appendix F for photographs)), as a result of high lake levels coupled with either rain or snow. Similarly, flooding caused by unusually high groundwater levels and spring-fed flows could be worsened by the presence of weed in the channel.

Therefore, for the efficient movement of water in the drainage system, an effective programme for the control of aquatic/drain bank weeds and silt removal is required. Aquatic weeds and silt build-up contribute to inefficient water use by restricting water movement within the drains, thus decreasing the drainage ability. Vegetation growth (perennial grasses), however, is desirable on drain banks at the waterline and above waterline to minimise the establishment of land weeds and to prevent bank, wind and water erosion (Shaw 2006).

Current drain cleaning aims to only remove the vegetation growing on the banks of the drain that restricts water flow and silt. Drain cleaning also removes the sand and weed that has been deposited in the bottom of the drain, which reduces the waterway area or the water velocity. This process of drain cleaning is currently carried out using a combination of mechanical cleaning, spraying and cutting (see Appendix G for more details).

Weed cutting only occurs on the L2 River. Cleaning normally starts in late January and takes about 12 days. Cleaning is carried out using a floating weed cutting plant referred to as the Ellesmere Queen (Figure 6-7). The weed cutting within the L2 River is usually triggered when the water level reaches 2.428 m asl for site 2 and 1.830 m asl for site 3 (C Hill, Chairman of the L2 drainage committee pers. Comm. 2007) (traditional levels) during the dry season. It is also important to note that during summer, when the lake level is low, swans found in the lake may move into the L2 River to eat the weeds, resulting in no weed cutting being carried out for that year (C Hill, Chairman of the L2 drainage committee pers. Comm. 2007). The cutting of the submerged weed within the L2 channel trims the semi-emergent species, which will in itself have a beneficial effect on reducing flooding. Resource consents relating to the installation of a boom to retain weed from the weed cutting process in the L2 were obtained in 1999 (Appendix H).



Figure 6-7 Ellesmere Queen, the floating weed cutting plant (Lane 2004)

Drain cleaning of the drainage ditches within the district is normally undertaken during the period of April/May. In the event that a major rainstorm event occurs during late spring or summer (i.e. when the weeds are blossoming) the drainage network performance would be severely affected, since both the culverts and drainage ditches within the network would be blocked with weeds (Figure 6-8).



Figure 6-8 Weeds within the drains at McDonald and Springs Road (January 2007)

Drain cleaning operations have been carried out intensively over the years; this has resulted in a deepened channel with steep sides and in the physical removal of stream life. As a result of this, the habitat has been damaged and this has also increased sediment following the cleaning operation from the steep sides of the bank collapsing (Figure 6-7).



Figure 6-9 Bank collapse due to the steep sides on bend within the L2 channel

6.2 Simulations of the Impact of Lake Level Rise coupled with large Rainstorm Events

The aim of the computational modelling programme was to quantify what the effects of higher lake levels, coupled with large storm events (2, 10 and 20 ARI) will be on the L2 drainage scheme in the future. To determine these effects, simulations using both the hydrologic and hydraulic models were executed for a number of scenarios. The HEC-HMS

model was used to simulate rainstorms with 2, 10 and 20 years Average Recurrence Interval (ARI) data for both the winter (August) and summer (December) periods. The results from the HEC-HMS simulation were then used as input for the hydraulic simulations of the L2 River with the MIKE 11 model. The hydraulic simulations were modelled with varying lake levels to determine what effects higher lake levels coupled with large rainstorms would have on the drainage network.

6.2.1 Hydrologic Simulation of Various Scenarios

6.2.1.1 User specified design storm

Rainfall data to calculate precipitation input for the hydrologic simulations was obtained from the HIRDS (High Intensity Rainfall Design System) software from NIWA (National Institute of Water and Atmospheric Research). Using the coordinates of the catchment center (5727828.55N; 2463992.70E (New Zealand Map Grid)), the rainfall depths were produced for 2, 10 and 20 year ARI storms. These data were then interpolated and analysed to obtain the redistributed rain (mm) so as to have maximum precipitation at the end of the second quartile. This was done based on rainfall data obtained for the L2 catchment and is a representation of storm events that are occurring within the catchment. The interpolation and analysis was done using the procedure outlined below. The natural log (\ln) of intensity (mm/hr) was plotted against \ln of duration (hrs) (Intensity Duration Frequency curve (IDF)) and a trend line was fitted to the data (Figure 6-10). From the obtained equation, rainfall intensity was computed for 12 hours in 30-minute intervals, since it is this standard for the region. From the intensity, accumulated depths (intensity times duration) and incremental depth (acc. depth at time, t minus acc. depth at time, $t-1$) were calculated and the latter was redistributed to have maximum precipitation at the end of the second quartile (Figure 6-11).

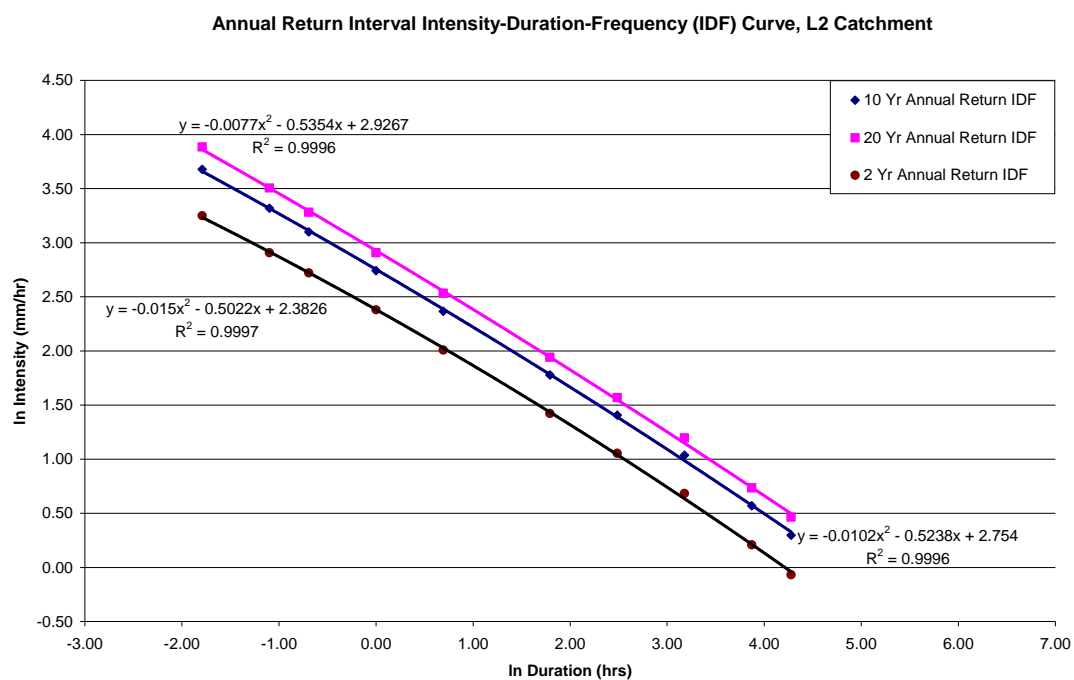


Figure 6-10 IDF curve, natural log of Intensity as a function of natural log of Duration for the 2, 10 and 20 year ARI storm events

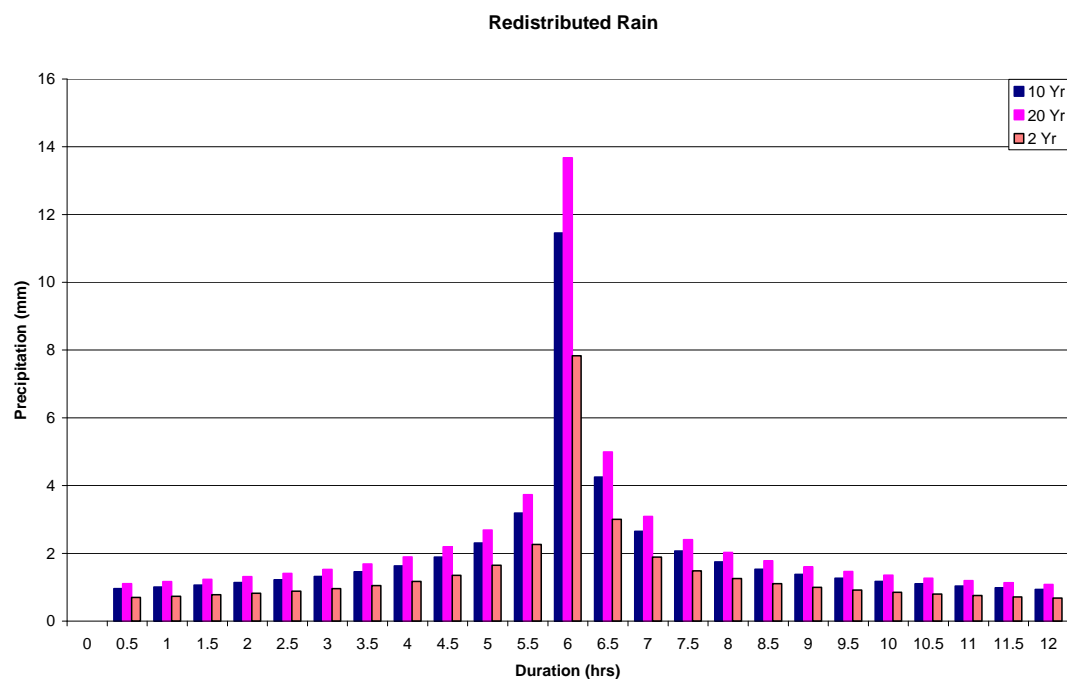


Figure 6-11 Redistributed precipitation using the second quartile method for the 2, 10 and 20 year ARI storm events

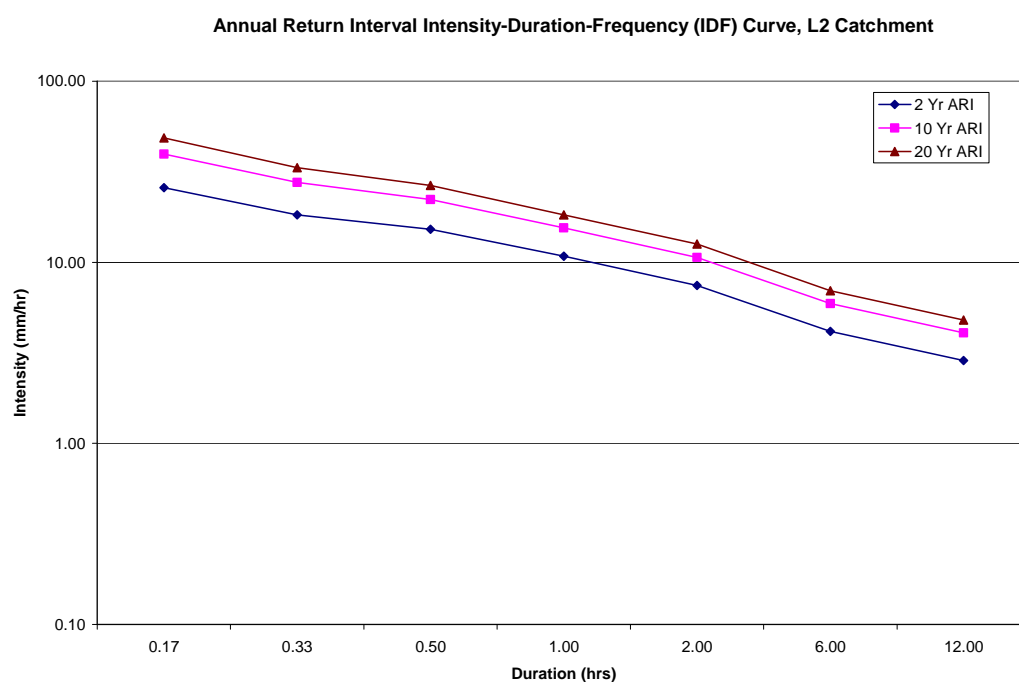


Figure 6-12 Rainfall intensity as a function of duration for the L2 catchment

All hydrologic simulations were run for 60 hours from the start of the precipitation. The 2, 10 and 20 year ARI storms were simulated for both winter (August month) and summer periods (December). Figure 6-13 and Table 6-1 shows a summary table of the results for the runs. There was a difference in the results for the simulations carried out between the winter and summer periods, which was due to the difference in baseflow observed within the catchment (see section 4.1.4.2).

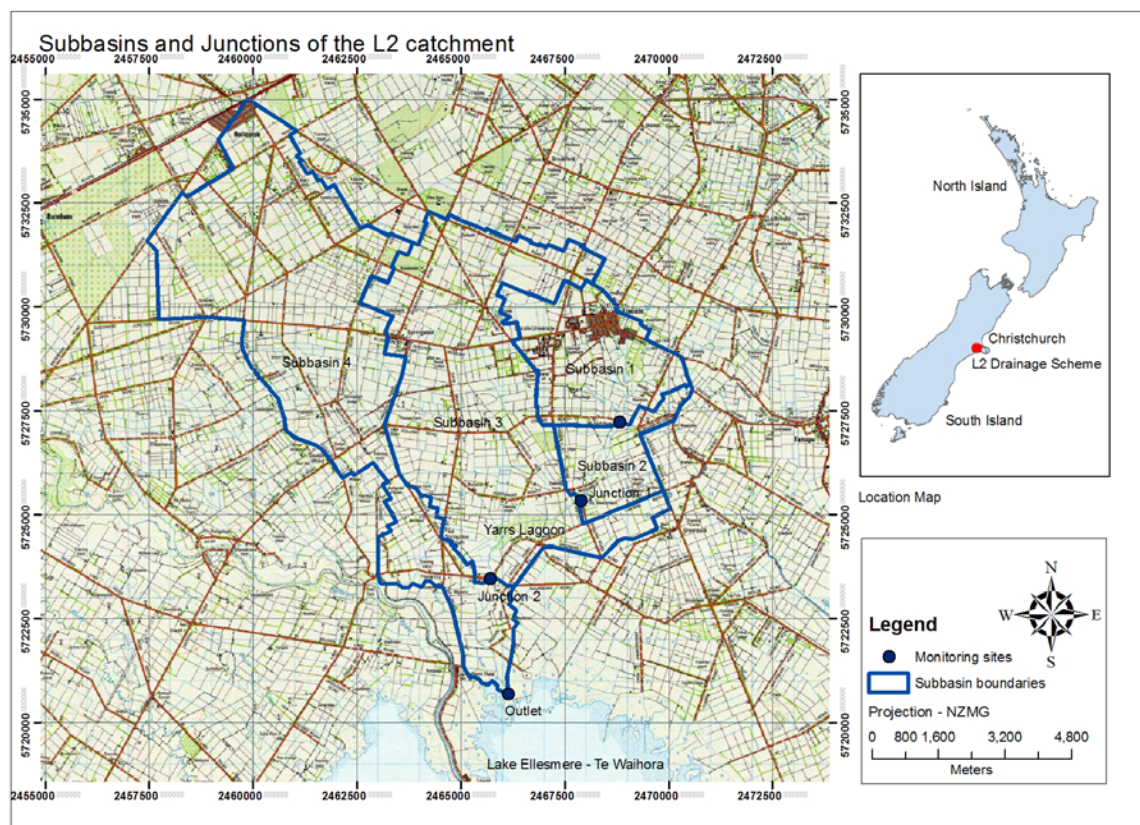


Figure 6-13 Subbasins, junctions and outlet of the L2 catchment (Land Information New Zealand 1:50000 Topographic Map sheets)

Table 6-1 Summary table of hydrologic simulations for current conditions for both winter and summer period

Winter							
	Peak Outflow (m ³ /s)						
ARI	Subbasin 1	Subbasin 2	Subbasin 3	Subbasin 4	Junction 1	Junction 2	Outlet
2 Yr	3.04	1.40	2.35	3.07	3.83	5.41	7.93
10 Yr	4.52	2.31	4.17	4.64	5.62	8.18	11.81
20 Yr	5.56	2.93	5.51	5.79	6.87	10.18	14.64

Summer							
	Peak Outflow (m ³ /s)						
ARI	Subbasin 1	Subbasin 2	Subbasin 3	Subbasin 4	Junction 1	Junction 2	Outlet
2 Yr	2.71	1.51	1.99	2.76	3.62	4.84	7.06
10 Yr	4.21	2.43	3.83	4.35	5.43	7.65	10.99
20 Yr	5.25	3.05	5.17	5.50	6.68	9.65	13.82

The winter storms produced a higher peak outflow than the summer storms by an average of 9.16 per cent, 5.12 per cent and 4.01 per cent for the 2, 10 and 20 year ARI storms respectively. Figures 6-14, 15 and 16 show the outflow from all basins, the junctions and the outlet for the 2, 10 and 20 year storms for the winter period. Apart from showing the specific results from the run, the graph also shows the flows in each part of the catchment and when the different peak flows occurred. From the beginning of the rainfall it takes approximately 27 hours before the peak outflow is visible at the outlet, whilst the runoff from the lower basin reaches the outlet earlier. For the 20 year ARI storm during the winter period, the peak flow at the outlet ($14.64 \text{ m}^3/\text{s}$) was 3.6 times larger than the baseflow ($4.08 \text{ m}^3/\text{s}$).

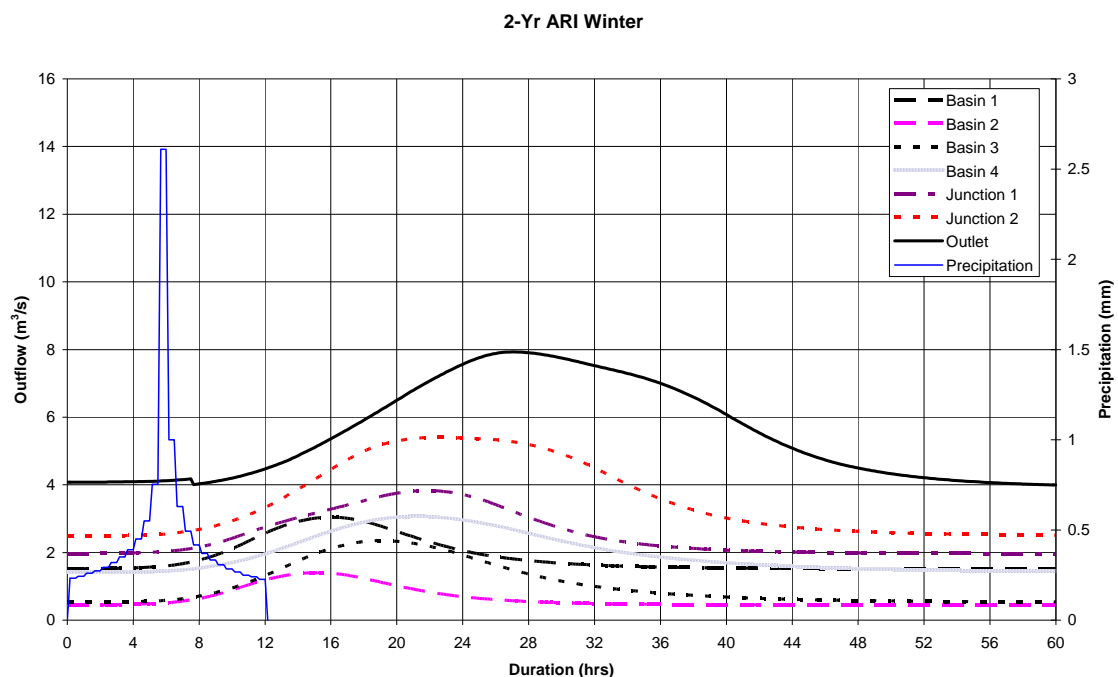


Figure 6-14 Simulation of the 2 year ARI storm event during winter period

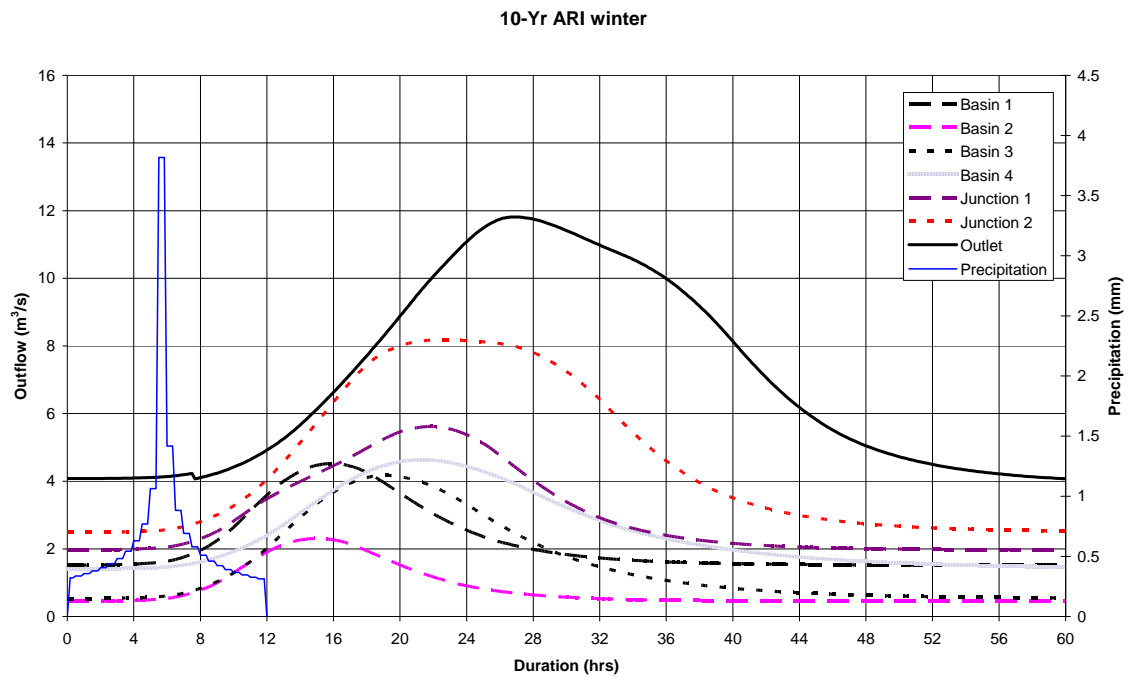


Figure 6-15 Simulation of the 10 year ARI storm event during winter period

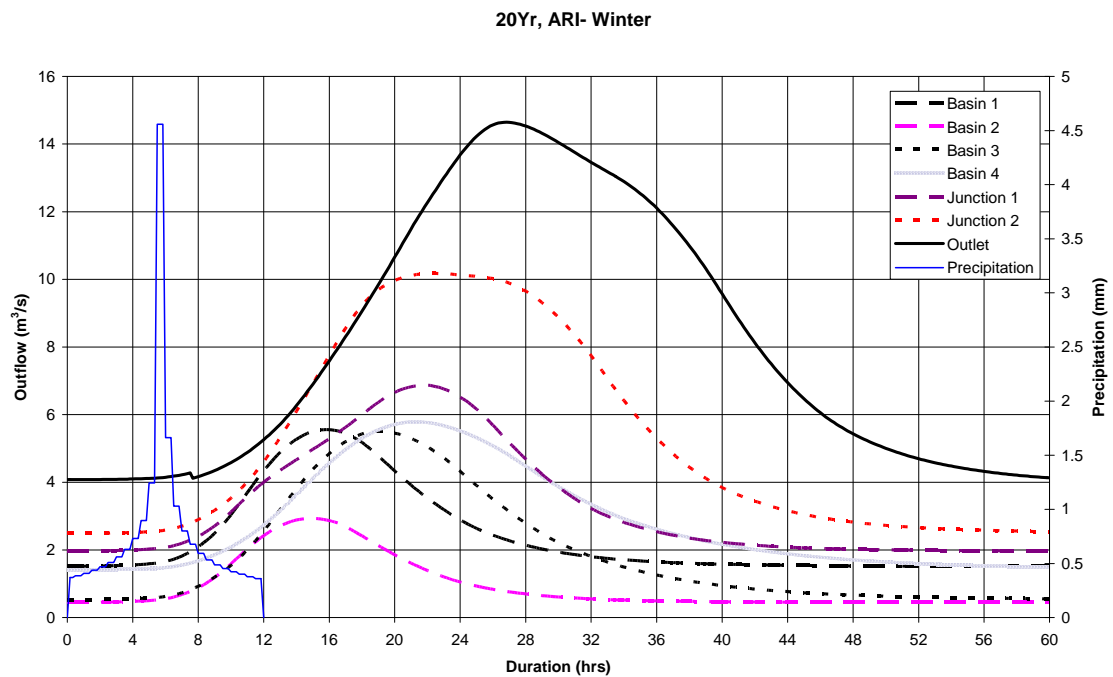


Figure 6-16 Simulation of the 20 year ARI storm event during winter period

For all the storms simulated for both winter and summer periods, the peak flow at the outlet was 161.8 per cent greater than the peak outflow from subbasin 1. It is believed that due to climate change the frequency of higher intensity storms will increase.

6.2.2 Hydraulic Simulation of Various Scenarios

The results from the hydrologic simulations were used as input for the hydraulic simulation. Hydraulic simulations were carried out for both winter (i.e. when the vegetation impact is minimal) and summer (i.e. when the vegetation is blossoming). For each storm event the lake level was set to 'normal' opening level (i.e. 1.13 m for winter and 1.05 m for summer), a 'high' lake level (i.e. 1.28 m for winter and 1.20 m for summer were assumed based on a 15 cm rise in lake opening level) and a 'very high' lake level (i.e. 1.48 m for winter and 1.40 m for summer were assumed based on a 35 cm rise in lake opening level). Figures 6-17, 18 and 19 show a summary of the simulated water levels for the 2, 10 and 20 year ARI storm events for 'normal', 'high' and 'very high' lake conditions for both winter and summer periods.

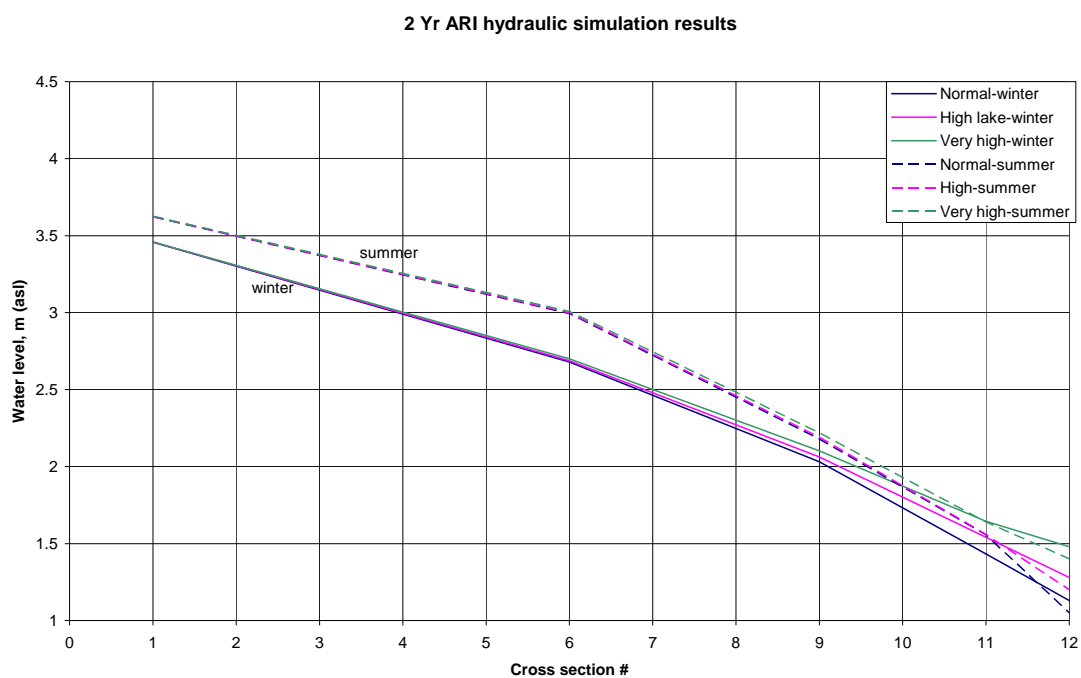


Figure 6-17 Summary of simulated results for the 2 year ARI storm event for winter and summer periods

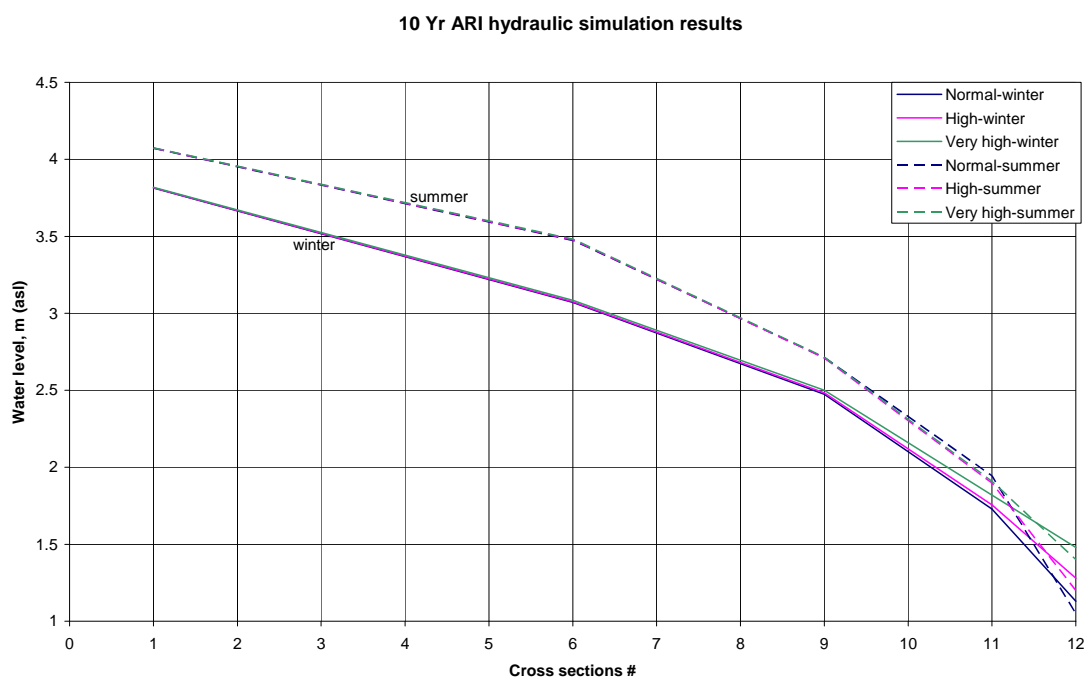


Figure 6-18 Summary of simulated results for the 10 year ARI storm event for winter and summer periods

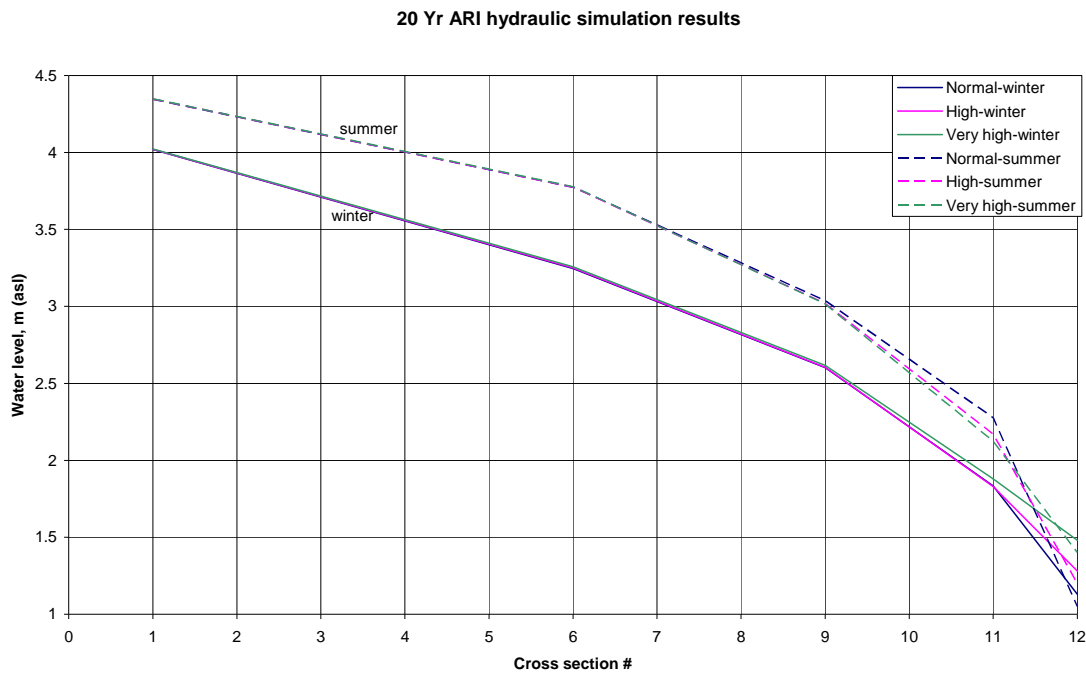


Figure 6-19 Summary of simulated results for the 20 year ARI storm event for winter and summer periods

Even though the winter simulated storms (hydrologic) produced larger peak outflows, the summer storms' simulations resulted in higher water levels within the channels, hence again highlighting the effect of the weeds within the channel. The peak outflow (m^3/s) for the two year ARI rainstorm, for the summer period was 9.16 per cent less than that for the corresponding storm during the winter. However, the water levels for the summer hydraulic simulation were found to be 7.95 per cent, 7.50 per cent and 7.25 per cent for the normal, high and very high lake levels respectively higher than that for the corresponding winter simulation. This trend continued, that is, the water level within the channel was greater during the summer simulations, than for winter simulations for both the 10 and 20 year ARI storms (Figures 6-17, 18 and 19).

Figure 6-20 shows the difference in water levels (dashed red lines) for both winter and summer periods for the 10 year ARI storm, with very high lake levels. The solid black lines represent the right bank (looking upstream), while the broken black line represents the left bank of the L2 River. The dashed red lines represent the maximum water level within the channel. As shown in Figure 6-20 (top), cross-sections 11 and 12 were completely flooded, that is, flooding occurred from chainage 6950 m (both banks were overtopped) to site 4. However, for the summer simulation, six cross-sections were completely overtopped (sections 1, 2, 5, 7, 11 and 12), with flooding occurring downstream from chainage 6300 m to site 4 (approximately 2.7 km from site 4), that is, a difference of 650 m more upstream for the summer simulation than for the winter (greater flooding upstream in the summer).

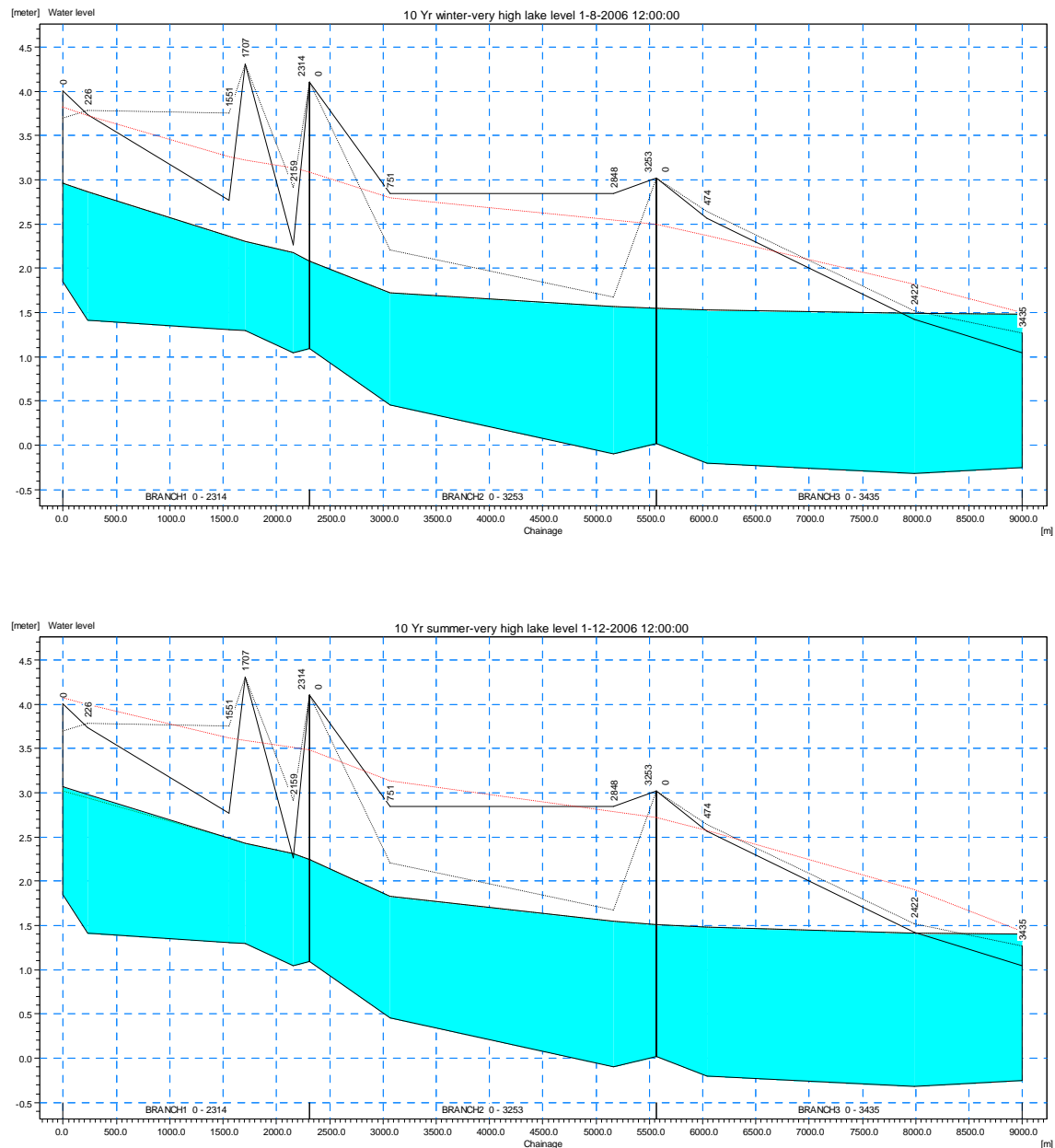


Figure 6-20 Comparison of water levels (dashed red lines) for both winter (top) and summer (bottom) periods for the 10-year storm with a very high lake level

Figure 6-21 (top) shows the simulated flood depth for the 20-year ARI rainstorm during the winter period at a very high lake level. Cross sections 1, 2, 5, 7, 11 and 12 (Figure 6-21 (bottom)) was flooded as a result of this storm coupled with a very high lake level. The maximum flood depth occurred at cross section 5 (Silverstream) with a depth which was

greater than 1.04 m. The second highest flood depth along the length of the L2 River occurred at Yarrs Lagoon (i.e. between cross sections 7 and 8) followed by cross section 3 (north of McDonald Road). These are the same areas identified previously that are likely to be flooded (Figure 5-6).

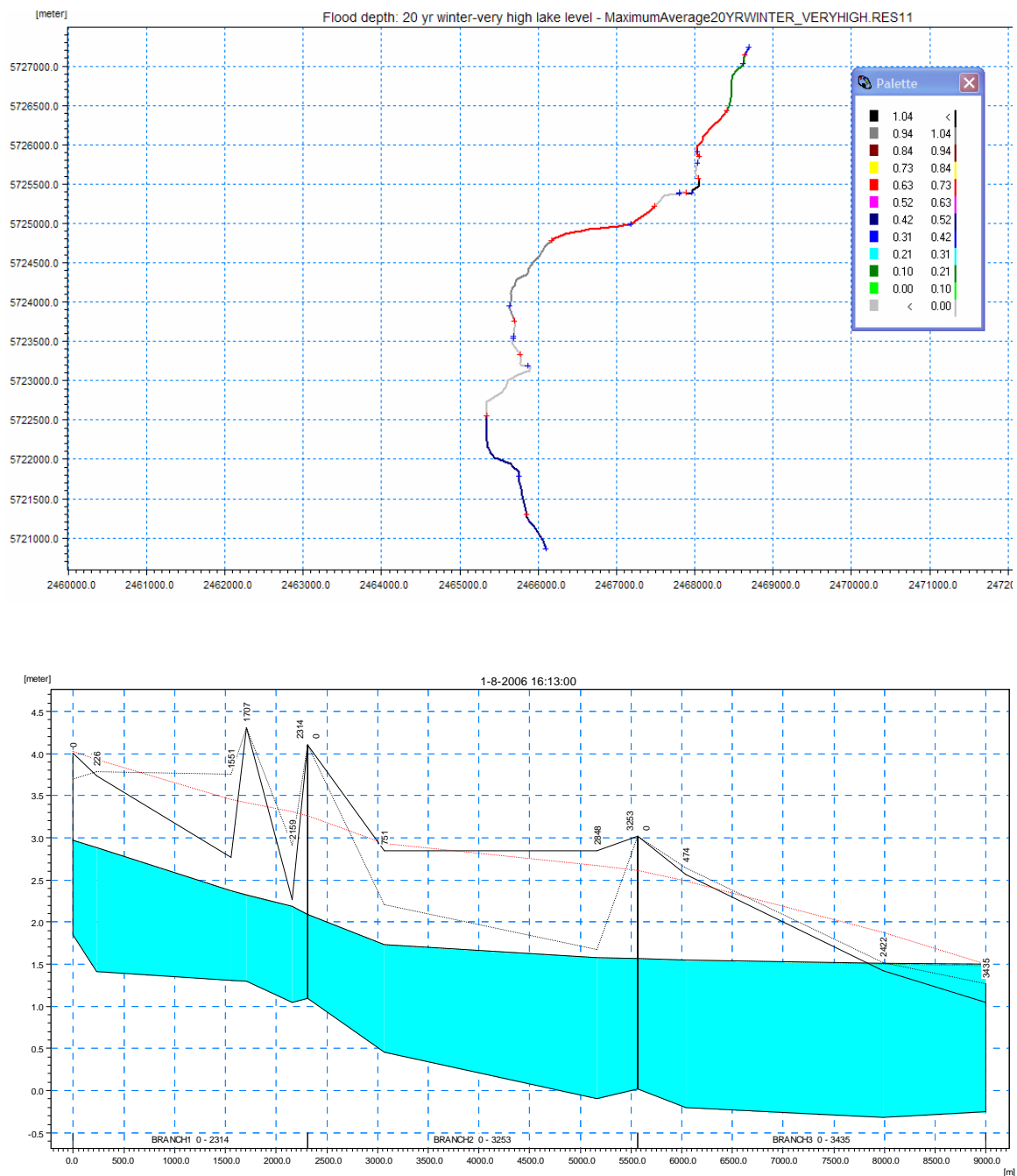


Figure 6-21 Flood depth (top) and the longitudinal view a 20-year ARI storm simulated during the winter period with a very high lake level

The increase in lake levels does not have much of an effect upstream of the L2 River, with the largest increase in water level of 3.50 per cent occurring at site 3 during the 2 year ARI storm event, with a 'very high' lake level during the winter period. The effect of the increased lake level is further reduced upstream, with site 1 registering a maximum increase in water level of 0.08 per cent and site 2 an increase of 0.78 per cent for all the various scenarios. Since the outlet (site 4), is influenced by the lake level, this area and approximately 2.7 km upstream of the river outlet is affected the most by the higher lake levels. With the lake level set to 'high' condition for the two year ARI storm, the entire section between cross-sections 11 and 12 was flooded.

In reality this means that farmlands adjacent to Lake Ellesmere will be flooded with an increase in lake opening levels for a 2 year ARI storm event. If the lake opening level is increased for both summer and winter to the 'high' level or the 'very high' level, the winter period will see the greatest percentage increase in the water level within the L2 River at baseflow conditions.

Even though the hydrologic model simulated large peak flows for the 2, 10 and 20 year ARI storms, in reality, the L2 River would not see such large peak flows and high water levels as simulated by the hydraulic model, simply because of the amount of ponding/flooding that takes place within the catchment. As shown in Figure 6-21, long stretches of the L2 River would overtop its banks causing flooding.

Even though a 'higher' lake level is required to maintain the same outflow gradient to the sea (due to the expected rise in sea level), this increase will not have any major effect upstream of the river, that is, from cross sections 10 and below (9, 8, 7...). The lower reaches of the L2 River would flood directly from higher lake levels. Therefore, the effect

from increased lake levels is not translated upstream in the L2 River and hence in the drains discharging into the drainage network.

In the event that the lake opening levels are increased and whenever a strong south-easterly wind is prevailing, the water level at the outlet of the L2 River will rise as much as 0.5 m. This sudden increase in water level at the outlet will only have a minimal effect upstream of the river as indicated by the hydraulic model and also by the water level data collected (Figure 4-5), but will result in flooding of the farmlands adjacent to Lake Ellesmere (within 2 km of the L2 River mouth). Therefore, measures would have to be implemented in order to prevent this flooding, which would be increased in frequency once the lake opening levels have been increased.

Since both the hydrologic and the hydraulic models performed satisfactorily, they can be used as a predictive tool for the management and operation of both Lake Ellesmere and the L2 drainage network.

7 Summary and Conclusions

7.1 Introduction

The potential impact of sea level rise on Lake Ellesmere – Te Waihora and the subsequent effect on the efficiency and performance of the L2 Drainage network were investigated in relation to the operation of the L2 Drainage scheme. Lake Ellesmere is currently manually opened for drainage to the sea when the lake levels reach 1.05 m above mean sea level (asl) in summer and 1.13 m asl in winter. Mean sea levels around New Zealand have already shown a 10 cm to 20 cm rise over the past 100 years and it is expected to rise between 9 to 88 cm by the end of the century⁶ (Ministry for the Environment). With a rise in sea level, the lake opening levels for both summer and winter would have to increase in order to maintain the current hydraulic gradient.

7.2 Objective and Methodology

The general objective for this thesis was to understand the flow dynamics of the drainage network so that the behaviour of the L2 drainage system could be predicted as Lake Ellesmere levels fluctuate. This research was carried out using an integrated approach to study this potential impact, including fieldwork, analysis of data, hydrologic and hydraulic modelling. Extensive fieldwork was necessary to measure the L2 River bed slope, cross-sectional geometries, channel roughness coefficient (Manning's n), flow velocities and water levels to enable calibration and modelling with the computational (hydrologic and hydraulic) models.

⁶ Extreme values from Ministry for the Environment report

7.3 Computational Modelling

Both the hydrologic and hydraulic response of the L2 catchment were reproduced with reasonable accuracy by computational models. The models were calibrated and validated by modifying input parameters in an attempt to match field observation (data collected and analysed from fieldwork). Four parameters: curve number, initial abstraction, impervious and lag time were subject to calibration and validation within the HEC-HMS hydrologic model. The MIKE 11 hydraulic model was calibrated and validated by modifying Manning's n values according to field observation.

7.4 Results of the Computational Modelling

The aim of the computational modelling was to quantify what the effects of higher lake levels, coupled with large storm events (2, 10 and 20 Average Recurrence Interval (ARI)) will be on the L2 drainage scheme in the future. To determine these effects, simulations using both the hydrologic and hydraulic models were executed for a number of scenarios. The HEC-HMS model was used to simulate rainstorms with 2, 10 and 20 years Average Recurrence Interval (ARI) data for both the winter (August) and summer (December) periods. The results from the HEC-HMS simulation were then used as input for the hydraulic simulations of the L2 River with the MIKE 11 model. The hydraulic simulations were modelled with varying lake levels to determine what effects higher lake levels coupled with large rainstorms would have on the drainage network.

An increase in the lake opening levels, coupled with south-easterly wind was shown to have increased the degree of flooding on adjacent farmlands, but only produced a 3.50 per cent increase of water level (maximum for all conditions simulated) 3.5 km upstream on the L2 River. The effect of the increased lake level is further reduced upstream, with monitoring

site 1 (9 km upstream of the river outlet) registering a maximum increase in water level of 0.08 per cent and site 2 (6.7 km upstream of the river outlet) an increase of 0.78 per cent for all the various scenarios. It was also found that the weeds within the L2 River control the water level within the channel, a rise of approximately 0.30 m was observed from the winter to summer season. Water level rise within the L2 channel has been associated with the weed growth. Even though the winter simulated storms (hydrologic) produced larger peak outflows, the summer storms' simulations resulted in higher water levels within the channels, hence again highlighting the effect of the weeds within the channel. The actual stream banks height determines whether the water level rise caused by the weeds is sufficient to cause flooding of surrounding land. Sections of the river with low stream bank height are susceptible to frequent overtopping.

7.5 Limitations in Measurements

Two major limitations affected the accuracy of this research. Firstly, the capacitance water level logger installed at monitoring site 1 did not capture the major rainstorm events that occurred during the period of study. This affected the calibration of the hydrologic model to some extent (measures were taken to overcome this problem). Secondly, the accuracy of the rating curves constructed to estimate discharge from the water level data recorded by the loggers was also affected due to the rapid growth of weeds from the winter to summer period within the channel.

7.6 Conclusions

Overall, the results of this study add to the understanding of the dynamics of the flow within the L2 River. The potential for both models to be used as a predictive tool for improving the operation of the L2 scheme and Lake Ellesmere was limited by the difficulty in estimating

model parameters, especially for the hydrologic model. However, with appropriate parameterization they are likely to be useful for both the operation of the river and Lake Ellesmere with its current control system, and as a basis for increasing their efficiency.

It was concluded that:

- The hydrologic and the hydraulic models that were developed can be used as predictive tools for the management and operation of both Lake Ellesmere and the L2 drainage network.
- The L2 drainage system was found to be efficient in draining water from a rainstorm event (not an extreme event), especially during the winter period, since drain cleaning is done prior to this season; however the efficiency is drastically reduced during the summer period, caused by the weed growth. In the event that there is a major rainstorm during the dry season, there will be lots of ponding/flooding areas within the scheme.
- It was found that with the sea level rise, the current lake opening levels would have to be increased to maintain the same outflow gradient to the sea.
- An increase in lake opening levels would have minimal water level impacts in the upstream reaches of the L2 River; the majority of the impacts would be seen on farmlands adjacent to the lake. With strong prevailing south-easterly winds this effect will be amplified.
- The water levels within the L2 River are drastically affected by weeds, by blocking the channel, hence effectively reducing the channel geometry and increasing the water level within the channel. The water velocity for all the monitoring sites

reduced over the period of study as a result of the vegetation growth within the channel, hence increasing the Manning's n coefficient from winter to summer.

- The sensitivity analysis of the hydraulic model shows the Manning's n was a sensitive parameter, but not a critical one. The least-defined cross section in the upper reach of the L2 River affects the elevation and the slope of the channel, and results in an increase in the water level within the channel. Likewise, since the lower reach is wider, fewer points can be surveyed. It was found that this affects the simulated water level to a small extent.
- The sensitivity analysis of the hydrologic model shows the most sensitive parameter was the curve number on the peak flow. The curve number was also the most sensitive parameter when it comes to total volume. The lag time was the most sensitive parameter when it comes to peak time.

8 Recommendations

Recommendations for further research:

- Improvement of the rating curves used to convert the measured water level to discharge, hence improving the calibration of the models.
- Further research into effects of weed within the L2 channel.
- Research into future land use changes including more drains, that is, from the establishment of new sub-divisions within the L2 catchment (varying curve numbers).
- Increase the number of cross-sections defined, as well as defining the drainage ditches draining into the L2 River, thus extending the scope of the hydraulic model.
- Study means of the protection of the adjacent farmland from flooding that will result from higher lake levels – while protecting the environment.
- Study the potential rise in soil moisture in farmland due to higher water levels in drains.
- To study the impact of groundwater within the catchment coupled with an increased lake level (a higher water table).
- Investigation of the water quality and the impact on ecology/fish as a result of an increased lake opening level from the lake being flushed less frequently.

Recommendations on how to manage a sea level rise:

- Investigation into increasing the storage capacity along the L2 channel (Yarrs Lagoon and probably Moirs Lane) as well as the development of wetlands and retention ponds to reduce the magnitude of flooding from extreme rainstorm events.
- Given more frequent extreme events that drain cleaning is done more often and more thoroughly within the catchment.
- The planting of appropriate vegetation (perennial grasses and others) on the slopes of the banks should be considered to prevent erosion due to high water levels or quicker fluctuations.

References

- Chanson, H. (1999). *The Hydraulics of Open Channel Flow: An Introduction: Basic principles, sediment motion, hydraulic modelling, design of hydraulic structures*, John Wiley & Sons Inc., New York.
- Chow, V. T., Maidment, D. R., and Mays, L. W. (1988). *Applied hydrology*, McGraw-Hill, New York.
- Cunge, J. A., Holly, F. M., and Verwey, A. (1980). *Practical Aspects of Computational River Hydraulics*, Pitman Advanced Publishing Program, Boston.
- Dark, A. L. (2005). "Flood Wave Travel Time in the Upper Tekapo River," University of Canterbury, Christchurch.
- Dataflow Systems Pty Ltd. *PC Software & Technical Handbook for Odyssey Data Recorders*, Christchurch.
- DHI Water and Environment. (2004). *MIKE 11: A modelling system for Rivers and Channels - User Guide*, DHI Water and Environment.
- Haslam, S. M. (1978). *River Plants: The Macrophytic Vegetation of Watercourses*, Cambridge University Press, Cambridge.
- Henderson, F. M. (1966). *Open Channel Flow*, Collier Macmillan Publishers, London.
- Hicks, D. M., and Mason, P. D. (1991). *Roughness Characteristics of New Zealand Rivers*, Water Resources Survey: DSIR Marine and Freshwater, Wellington.
- Horrell, G. A. (1992). "Lake Ellesmere Water Model: Variable analysis and Evaluation," University of New South Wales, Sydney.
- Hudson, H. R. (2005). "H2O-DS Hillslopes to Oceans: Decision Support System for Sustainable Drainage Management." *New Zealand Water Environment Research Foundation*, 150.
- Hunt, B. (1995). *Fluid Mechanics for Civil Engineers*, Department of Civil Engineering, University of Canterbury, Christchurch.

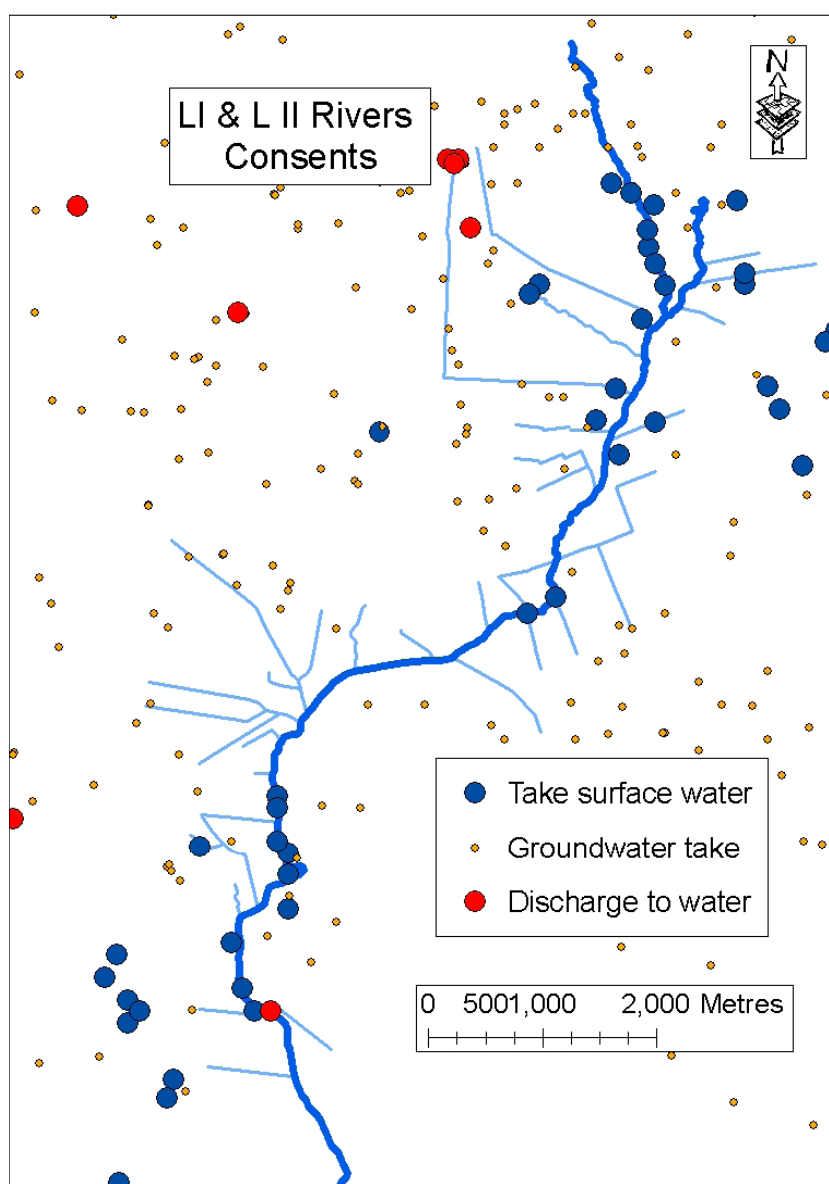
- Iremonger, A. (2005). "Land Drainage Schemes Operation Manual: Draft for Discussion." 64-057-1011, Selwyn District Council, Christchurch.
- Kouwen, N. (1988). "Field estimation of the Biomechanical Properties of grass." *Journal of Hydraulic Research*, 26(5), 559-568.
- Kouwen, N., Unny, T. E., and Hill, H. M. (1969). "Flow Retardance in Vegetated Channels." *J. Irrig. Drain. Div.*, ASCE 95((IR2)), 329-342.
- Lane, D. (2004). "L1 & L11 Catchment Report." Environment Canterbury, Christchurch.
- Maw, R. (2004). "Draft Minimum Flow Recommendations for: L2 Catchment." Environment Canterbury, Christchurch.
- Mays, L. W. (2005). *Water Resources Engineering*, John Wiley & Sons, Inc., New Jersey.
- Ministry for the Environment. (2006a). "Climate Change: Impacts on New Zealand." Ministry for the Environment Wellington.
- Ministry for the Environment. (2006b). "Helping New Zealand Prepare for and Adapt to the Impacts of Climate Change." M. f. t. Environment, ed., 4.
- Morris, J., and Dunderdale, J. A. L. (1996). "The Economics of Aquatic Vegetation removal in Rivers and Land Drainage Systems." *Hydrobiologia*, 340, 157-161.
- Mosley, M. P. (1992). "Waters of New Zealand." Streamflow and the Use of Models, New Zealand Hydrological Society Inc., Wellington.
- NIWA. (2003). "Angler usage of Lake and River Fisheries managed by Fish & Game New Zealand: Results from the 2001/02 National Angling Survey." *CHC2003-114*, Christchurch.
- Petryk, S., and Bosmanjian, G. I. (1975). "Analysis of Flow through Vegetation." *J. Hydraul. Div.*, ASCE 101(HY7), 871-884.
- Ponce, V. M. (1989). *Engineering Hydrology: Principles and Practices*, Prentice-Hall, Englewood Cliffs, NJ.

- Ricon, P. (2006). "Sea rise could be 'Catastrophic'." *Science/Nature*, London.
- Righetti, M., and Armanini, A. (2002). "Flow Resistance in Open Channel Flows with Sparsely Distributed Bushes." *Journal of Hydrology*, 269(2002), 55-64.
- Rosen, M. R., and White, P. A. (2001). "Groundwaters of New Zealand." New Zealand Hydrological Society Inc., Wellington.
- Scharffenberg, W. A., and Fleming, M. J. (2006). *Hydrologic Modelling System HEC-HMS User's Manual*, Washington.
- Shaw, H. (2006). "Lincoln Integrated Catchment Plan." 42159587/6000, Selwyn District Council, Christchurch.
- Stephan, U., and Gutknecht, D. (2002). "Hydraulic Resistance of Submerged Flexible Vegetation." *Journal of Hydrology*, 269(2002), 27-43.
- Taylor, D. (2005). "Rural Drainage Management – Decision Analysis for Best Practice." University of Canterbury, Christchurch.
- Taylor, K. J. W. (1996). "The Natural Resources of Lake Ellesmere (Te Waihora) and its Catchment." 96(7), Canterbury Regional Council, Christchurch.
- Wong, C. W. (1999). "Development of an Integral Framework for Environmental Decision Making," Lincoln University, Christchurch.

Appendices

A. Groundwater abstraction, surface water takes and discharge consents along the L2 River

This appendix shows groundwater abstractions, surface water takes and discharge consents along the L2 River, obtained from Lane (2004).



Appendix A 1 Groundwater abstraction, surface water takes and discharge consents along the L2 River (Lane 2004)

B. Low Flow Recommendation

This appendix contains minimum low flow recommendations for the L2 River, obtained from the Lincoln Integrated Catchment Plan (ICMP) report compiled by URS consultants.

Draft Minimum Flow Recommendations for:

Halswell Catchment

Knights Creek at Jamieson's Property (a tributary of the Halswell)
Halswell River at Leadleys Bridge
Halswell River at Branthwaites Bridge
Halswell River at Ryans Bridge
Halswell River at Tobecks Bridge
Halswell River at Nellis Road

L II Catchment

L II at Moir's Property
L II at Pannetts Road

Kaituna Lagoon Catchment

Kaituna River at Kaituna Valley Road
Prices Stream at Prices Valley Road

Report prepared by Ray Maw
September 2004

Discussion Draft – Not Council Policy

L II Catchment

LII River – Moir's Property

Technical Panel Summary

Site Flows (L/s)		Suggested Minimum Flows (L/s)					
Gauged	7DMALF	Glova	Duncan	Taylor	Partridge	McManaway	Ngai Tahu
*	200**	350	200	200	250	350-400	450

* Unable to be gauged at time of assessment

** Recommended because it cannot be calculated

This is a deep spring-fed stream of low gradient with slow-flowing clear water. While suitable for short fin eel, the habitat is less desirable for trout due to lack of shade plants and the high level of emergent weeds. Traditional mahinga kai values related to eel habitat rather than food gathering because the area was once part of a deep swamp.

The stream has little sense of natural character at the observed site. There are no native vegetation species within the riparian margin. The is extensive growth of cress, floating lemma and musk along the observed reach.

Ranking Policy Criteria

The relative importance of the criteria identified in the RPS for ensuring sufficient quantities of water remain in water bodies were ranked as follows:

Policy 1 Ranking

Natural character	M-L
Mauri	L
Trout	L-M
Native fish	L
Mahinga kai	H (traditional)
Wāhi tapu & wāhi taonga	Not present
Indigenous vegetation	L

Policy 2 Ranking

Boat passage	L
Angling amenity	L
General amenity	M

Local Knowledge

The Advisory Group has provided the following information:

- o Constant flow fed primarily from three large springs and confined to one large property.
- o The health of the stream is good with plenty of eels, some trout and some flounder.
- o There are no surface abstractions.
- o Maintenance extremely significant - the amount of weed present determines change more frequently than flows
- o There are many big springs just across from Moir's Lane gauging point
- o There are springs all the way down. It is difficult to know how much water comes from Yarr's Lagoon, which is "like a big sponge"

Discussion Draft – Not Council Policy

Current Abstractions

At 01/03/04, the consents database showed three surface-water permits, taking a total of 105 L/s, referenced to this site. Two permits have minimum flows of 120 L/s while one has a minimum of 350 L/s.

Depending on when the above water permits were issued, any one of two different minimum flows is currently attached to the above permits. 120 L/s was assigned in accordance with the WSC Act 1967 or 350 L/s as recommended by the 2000 Expert Panel. For a fuller explanation see earlier commentary for Leadleys Road Bridge.

Staff Recommendation

Recommended Minimum Flow L/s	Current Minimum Flow L/s	7DMALF L/s
200 L/s	120 & 350	200
<p>Reasons</p> <p>The minimum flows put forward by the technical panel range from 200 to 450 L/s. There are no values that have been identified as highly important, apart from an historical mahinga kai value. Two panellists suggest a minimum flow of 200L/s is sufficient to protect native fish and hence mahinga kai. Such a flow would also provide a level of protection to amenity and natural character values in keeping with their importance.</p> <p>The three abstractions with minimum flows referenced to this site relate to tributary sites that are not gauged. Gauging this site is difficult but the lowest flow measured at this site is 216 L/s (11/09/01). The recommended minimum flow is below this lowest recorded flow. As a result, there is unlikely to be any significant impact on the two abstractors with 120 L/s restrictions. The abstractor with a current minimum of 350 L/s would benefit from the proposal. The proposed minimum does conflict with the local knowledge recorded.</p> <p>Therefore, the recommended minimum flow of 200 L/s adequately provides for the values set out in Objective 1, Chapter 9 of the RPS.</p>		

Lil River - Pannetts Road

Technical Panel Summary

Site Flows (L/s)		Suggested Minimum Flows (L/s)					
Gauged	7DMALF	Glova	Duncan	Taylor	Partridge	McManaway	Ngai Tahu
897	1824± 42	900	1000	900	0	900	1500

This is a stable gently flowing spring-fed stream meandering through farmland. The silted bed supports many submerged macrophytes although the environment is not ideal for benthic invertebrates. The stream possesses good holding water for large trout and an abundance of in-stream cover.

Discussion Draft – Not Council Policy

Clear water, gentle meanders and gently sloping banks contribute a reasonable sense of natural character. General amenity, including access, is good. Mahinga kai values are high in relation to habitat rather than resource use and the stream contributes significant freshwater input to Lake Ellesmere/Te Waihora. A big bed of Potamogeton is present at the observed site, along with the aquatic margin species of lemna, azolla, musk, cress and sweet grass. No native plant species are present in the riparian margin near this site.

Ranking Policy Criteria

The relative importance of the criteria identified in the RPS for ensuring sufficient quantities of water remain in water bodies were ranked as follows:

Policy 1 Ranking	
Natural character	M
Maori	M
Trout	H
Native fish	H
Mahinga kai	H
Wāhi tāpu & wāhi taonga	Not present
Indigenous vegetation	L

Policy 2 Ranking	
Boat passage	H
Angling amenity	H
General amenity	M

Local Knowledge

No information was made available from the Advisory Group.

Current Abstractions

At 01/03/04, the consents database showed there were no surface-water permits referenced to this site. Instead, a site approximately 1.5 kilometres downstream at Wolfes Road has five surface-water permits, taking a total of 76.7 L/s, referenced to it.

Staff Recommendation

Recommended Minimum Flow L/s	Current Minimum Flow L/s	7DMALF L/s
1000 L/s along with no further surface-water abstraction, or hydraulically connected groundwater abstraction, from the stream other than takes permitted by the Natural Resources Regional Plan* Note: this site would replace the Wolfes Road site.	None	1824± 42

Discussion Draft – Not Council Policy

Reasons

The minimum flows put forward by the technical panel range from 0 to 1500 L/s. Trout, native fish, angling amenity and mahinga kai values have been identified as highly important. Minimum flows of 900-1000 L/s have been suggested by the respective panellists as suitable for the protection of trout, native fish and angling amenity. Mahinga kai values have also been identified as highly significant while mauri is ranked as moderately significant. The respective panellist put forward 1500 L/s as a recommendation minimum flow to protect these values.

The trout, native fish and angling amenity values can all be adequately protected with a minimum flow of 1000 L/s. It is considered that such a minimum flow would also adequately provide for mahinga kai. Given the moderate ranking of mauri by the respective panellist, there is insufficient justification for the suggested 1500 L/s minimum flow. Therefore, a minimum flow of 1000 L/s is recommended.

Gauging flows at the Wolfes Road site presents major technical difficulties and an alternative site at Pannetts Road provides a compatible alternative. The recommended minimum flow is less than that currently assigned to the Wolfes Road site, with the exception of one take with an assigned minimum flow of 550 L/s. As a result, there would be a positive impact for all but one of the present abstractors. It is not considered that the impact of that abstractor is significant given that the likelihood of flows less than 1000 L/s is very low.

The L II is the single largest source of fresh water flowing into Lake Ellesmere/Te Waihora. At low flows and high lake levels, highly discoloured water of a higher salinity can penetrate further up from the mouth. Therefore, it is considered important that increased abstraction does not lead to a deterioration of the water balance of the Lake or the water quality of the lower reaches. Restricting any further surface-water and hydraulically connected ground-water abstractions would achieve that end without impact on current abstractors. Therefore, the recommended minimum flow should also carry this restriction proviso.

Therefore, the recommended minimum flow of 1000 L/s adequately provides for the values set out in Objective 1, Chapter 9 of the RPS.

* Note: The Proposed Natural Resources Regional Plan provides for the taking or diversion of up to 10 cubic metres of water per day per property, and at a rate not exceeding 5 L/s, for reasonable domestic or stockwater small scale use.

C. Loggers and Cross Sections Photographs

This appendix show photographs of the locations where water level loggers were installed as well as the locations where cross sections were surveyed along the length of the L2 River.



Appendix B1 Looking upstream of logger site 1/cross section 1 (Loch Ness)



Appendix B2 Capacitance water level logger (site 1)/Cross section 1



Appendix B3 Cross section 2



Appendix B4 Cross section 3



Appendix B5 Cross section 4



Appendix B6 Cross section 5



Appendix B7 Capacitance water level logger 2 (site 2)/Cross section 6



Appendix B8 Cross section 7



Appendix B9 Cross section 8



Appendix B10 Capacitance water level logger 3 (site 3)/Cross section 9



Appendix B11 Cross section 10



Appendix B12 Cross section 11



Appendix B13 Capacitance water level logger 4 (site 4)/Cross section 12

D. Runoff Curve Numbers Table

This appendix contains the runoff curve number table used for estimating the curve number for the L2 catchment, obtained from Mays (2005).

Table 8.7.3 Runoff Curve Numbers (Average Watershed Condition, $I_a = 0.2S$)

Land Use Description		Curve Numbers for Hydrologic Soil Group			
		A	B	C	D
Fully developed urban areas ^a (vegetation established)					
Lawns, open spaces, parks, golf courses, cemeteries, etc.					
Good condition; grass cover on 75% or more of the area		39	61	74	80
Fair condition; grass cover on 50% to 75% of the area		49	69	79	84
Poor condition; grass cover on 50% or less of the area		68	79	86	89
Paved parking lots, roofs, driveways, etc.		98	98	98	98
Streets and roads					
Paved with curbs and storm sewers		98	98	98	98
Gravel		76	85	89	91
Dirt		72	82	87	89
Paved with open ditches		83	89	92	93
		Average % impervious ^b			
Commercial and business areas		85	89	92	94
Industrial districts		72	81	88	91
Row houses, town houses, and residential		65	77	85	90
with lot sizes 1/8 acre or less					92
Residential: average lot size					
1/4 acre		38	61	75	83
1/3 acre		30	57	72	81
1/2 acre		25	54	70	80
1 acre		20	51	68	79
2 acre		12	46	65	77
Developing urban areas ^c (no vegetation established)					
Newly graded area		77	86	91	94
Cover					
Land Use	Treatment of Practice	Hydrologic Condition ^d			
Cultivated agricultural land					
Fallow	Straight row		77	86	91
	Conservation tillage	Poor	76	85	90
Row crops	Conservation tillage	Good	74	83	88
	Straight row	Poor	72	81	88
	Straight row	Good	67	78	85
	Conservation tillage	Poor	71	80	87
	Conservation tillage	Good	64	75	82

Table 8.7.3 Runoff Curve Numbers (*continued*)

Cover		Hydrologic Condition ^d	Curve Numbers for Hydrologic Soil Group			
Land Use	Treatment of Practice		A	B	C	D
Small grain	Contoured	Poor	70	79	84	88
	Contoured	Good	65	75	82	86
	Contoured and conservation tillage	Poor	69	78	83	87
	Contoured and conservation tillage	Good	64	74	81	85
	Contoured and terraces	Poor	66	74	80	82
	Contoured and terraces	Good	62	71	78	81
	Contoured and terraces	Poor	65	73	79	81
	Contoured and terraces and conservation tillage	Good	61	70	77	80
	Straight row	Poor	65	76	84	88
	Straight row	Good	63	75	83	87
	Conservation tillage	Poor	64	75	83	86
	Conservation tillage	Good	60	72	80	84
	Contoured	Poor	63	74	82	85
	Contoured	Good	61	73	81	84
	Contoured and conservation tillage	Poor	62	73	81	84
	Contoured and conservation tillage	Good	60	72	80	83
	Contoured and terraces	Poor	61	72	79	82
	Contoured and terraces	Good	59	70	78	81
Close-seeded legumes or rotation meadow ^e	Contoured and terraces	Poor	60	71	78	81
	Contoured and terraces and conservation tillage	Good	58	69	77	80
	Straight row	Poor	66	77	85	89
	Straight row	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	Contoured	Good	55	69	78	83
Noncultivated agricultural land Pasture or range	Contoured and terraces	Poor	63	73	80	83
	Contoured and terraces	Good	51	67	76	80
	No mechanical treatment	Poor	68	79	86	89
	No mechanical treatment	Fair	49	69	79	84
	No mechanical treatment	Good	39	61	74	80
	Contoured	Poor	47	67	81	88
Meadow	Contoured	Fair	25	59	75	83
	Contoured	Good	6	35	70	79
	Contoured	—	30	58	71	78
Forestland—grass or orchards—evergreen or deciduous	—	Poor	55	73	82	86
	—	Fair	44	65	76	82
	—	Good	32	58	72	79
Brush	—	Poor	48	67	77	83
	—	Good	20	48	65	73
Woods	—	Poor	45	66	77	83
	—	Fair	36	60	73	79
	—	Good	25	55	70	77
Farmsteads	—	—	59	74	82	86
Forest—range	—	—	—	—	—	—
Herbaceous	—	Poor	—	79	86	92
	—	Fair	—	71	80	89
	—	Good	—	61	74	84

Table 8.7.3 Runoff Curve Numbers (*continued*)

Cover		Hydrologic Condition ^d	Curve Numbers for Hydrologic Soil Group			
Land Use	Treatment of Practice		A	B	C	D
Oak-aspen		Poor		65	74	
		Fair		47	57	
		Good		30	41	
Juniper-grass		Poor		72	83	
		Fair		58	73	
		Good		41	61	
Sage-grass		Poor		67	80	
		Fair		50	63	
		Good		35	48	

^a For land uses with impervious areas, curve numbers are computed assuming that 100% of runoff from impervious areas is directly connected to the drainage system. Pervious areas (lawn) are considered to be equivalent to lawns in good condition and the impervious areas have a *CN* of 98.

^b Includes paved streets.

^c Use for the design of temporary measures during grading and construction. Impervious area percent for urban areas under development vary considerably. The user will determine the percent impervious. Then using the newly graded area *CN* and Figure 8.7.1a or b, the composite *CN* can be computed for any degree of development.

^d For conservation tillage poor hydrologic condition, 5% to 20% of the surface is covered with residue (less than 750-lb/acre row crops or 300-lb/acre small grain).

For conservation tillage good hydrologic condition, more than 20% of the surface is covered with residue (greater than 750-lb/acre row crops or 300-lb/acre small grain).

^e Close-drilled or broadcast.

For noncultivated agricultural land:

Poor hydrologic condition has less than 25% ground cover density.

Fair hydrologic condition has between 25% and 50% ground cover density.

Good hydrologic condition has more than 50% ground cover density.

For forest-range:

Poor hydrologic condition has less than 30% ground cover density.

Fair hydrologic condition has between 30% and 70% ground cover density.

Good hydrologic condition has more than 70% ground cover density.

Source: U.S. Department of Agriculture Soil Conservation Service (1986).

E. Sensitivity Analysis

This appendix contains summaries of the sensitivity analysis of the hydrologic model, for 3 rainstorm events within the period of August to October, 2006.

Appendix D 1 Sensitivity Analysis Summary Table – Curve Number

CN number	Original Values	Storm 1 Analyse Values	% change	Original Values	Storm 2 Analyse Values	% change	Original Values	Storm 3 Analyse Values	% change
Average curve number	63.000	76.250		63.000	76.250		63.000	76.250	
% difference to original value			21.032			21.032			21.032
Model Results									
Peak Value (m ³ /s)	4.940	4.980		5.640	6.230		21.130	30.950	
% difference to original value			0.810			10.461			46.474
Peak time (hrs after start)	32.500	32.830		29.500	30.330		39.170	38.830	
% difference to original value			1.015			2.814			-0.868
Total Volume (1000 m ³)	1211.350	1219.870		1152.700	1190.360		2698.150	3367.660	
% difference to original value			0.703			3.267			24.814
Relative sensitivity (% change results / % change CN)									
Peak value (%)			0.038			0.497			2.210
Peak time (%)			0.048			0.134			-0.041
Total volume (%)			0.033			0.155			1.180

Appendix D 2 Sensitivity Analysis Summary Table – Initial Analysis

I _a	Original Values	Storm 1 Analyse Values	% change	Original Values	Storm 2 Analyse Values	% change	Original Values	Storm 3 Analyse Values	% change
Average I _a number (mm)	12.700	33.628		12.700	33.628		12.700	33.628	
% difference to original value			164.785			164.785			164.785
Model Results									
Peak Value (m ³ /s)	4.940	4.900		5.640	5.220		21.130	12.720	
% difference to original value			-0.810			-7.447			-39.801
Peak time (hrs after start)	32.500	32.170		29.500	29.500		39.170	46.000	
% difference to original value			-1.015			0.000			17.437
Total Volume (1000 m ³)	1211.350	1203.220		1152.700	1118.200		2698.150	2030.710	
% difference to original value			-0.671			-2.993			-24.737
Relative sensitivity (% change results / % change I_a)									
Peak value (%)			-0.005			-0.045			-0.242
Peak time (%)			-0.006			0.000			0.106
Total volume (%)			-0.004			-0.018			-0.150

Appendix D 3 Sensitivity Analysis Summary Table – Lag Time

Lag Time	Original Values	Storm 1 Analyse Values	% change	Original Values	Storm 2 Analyse Values	% change	Original Values	Storm 3 Analyse Values	% change
Average lag time (mins)	612.500	705.000		612.500	705.000		612.500	705.000	
% difference to original value			15.102			15.102			15.102
Model Results									
Peak Value (m ³ /s)	4.940	4.900		5.640	5.580		21.130	20.280	
% difference to original value			-0.810			-1.064			-4.023
Peak time (hrs after start)	32.500	33.830		29.500	30.500		39.170	40.670	
% difference to original value			4.092			3.390			3.829
Total Volume (1000 m ³)	1211.350	1210.990		1152.700	1152.330		2698.150	2698.060	
% difference to original value			-0.030			-0.032			-0.003
Relative sensitivity (% change results / % change Lag Time)									
Peak value (%)			-0.054			-0.070			-0.266
Peak time (%)			0.271			0.224			0.254
Total volume (%)			-0.002			-0.002			0.000

F. Photographs – Flood Events

This appendix contains photographs of flood events that occurred in August 1986, August 1992 and July 1994, obtained from Environment Canterbury.

August 1986 Flood Event



Appendix E1 L2 River in foreground. Hudsons/Ellesmere Road junction centre left. (East) (1986-08-24)



Appendix E2 L2 River in centre. Collins road lower centre. (East) (1986-08-24)



Appendix E3 Selwyn River upper right. Days Road and South Springston L2 on left. (South) (1986-08-24)



Appendix E4 L2 in centre. Looking south across Yarrs Flat to Lake Ellesmere. (1986-08-24)



Appendix E5 Selwyn River upper right. Days Road and Tramway Road



Appendix E6 L2 River in centre. Springston south lower left. (East) (1986-08-24)

Photographs – August 1992 Flood Event



Appendix E7 Englishs Road and L2 lower centre (Southeast) (1992-08-30)



Appendix E8 L2 in foreground. Pannetts Road Bridge centre right (Southeast) (1992-08-30)



Appendix E9 L2 on left, Yarrs Flat on right (North) (1992-08-30)

Photograph – July 1994 Flood Event



Appendix E10 L2 at Lake Ellesmere (Southwest) (1994-07-28)

G. Drain Maintenance

This appendix contains summaries of drain maintenance carried out with the L2 catchment, obtained from the Lincoln Integrated Catchment Plan (ICMP) report compiled by URS consultants.

The day-to-day management of the Council's Land Drainage Systems is carried out by a mixture of Council's Drainage Committees, Asset Management Department and the Drainage Supervisor of EDS (Engineering Design Services, Councils Design and Contract Management Group). Cleaning of the drains is carried out using local contractors. Weed Cutting of L2 is carried out by the L2 Drainage Committee. Details of the management responsibilities for the individual schemes are detailed in Table D1 below.

Design functions are contracted through EDS as a result of Council agreement allowing EDS all Design and Contract supervision requirements for general operation and maintenance works.

Table D1 - Management Responsibilities

Scheme	Component	Cleaning Schedule	Cleaning Supervision	Spraying	Inspections	General Maintenance
Taumutu Drainage District	Drains	DC & SDC	SDC	SDC	SDC	SDC
Taumutu Culverts Drainage District	Drains	DC & SDC	SDC	SDC	SDC	SDC
	Culverts	DC & SDC	SDC	SDC	SDC	SDC
Leeston Drainage District	Drains	DC & SDC	SDC	SDC	SDC	SDC
Leeston Township Drainage District	Drains	DC & SDC	SDC	SDC	SDC	SDC
Ellesmere Drainage District	Drains	DC & SDC	SDC	SDC	SDC	SDC
LII Drainage District	Drains	DC	DC	DC	DC	DC
Greenpark Drainage District	Drains	DC & SDC	SDC	SDC	SDC	SDC
Osbornes Drainage District	Drains	DC & SDC	SDC	DC & SDC	DC & SDC	SDC
	Pump Station	-	DC	-	DC	DC
Wairiri Valley Drainage Area	Drains	-	-	-	-	SDC
Horarata River Drainage Area	River	DC & SDC	SDC	SDC	DC & SDC	SDC

Note: DC= Drainage Committee

Land Drain Monitoring

All land drains are to be regularly monitored to ensure appropriate standards are maintained to deliver the design flow.

- Critical drains are to be inspected at least once every year
- Minor drains are to be inspected at least once every five years.

This drain inspection is carried out by either the Drainage Committees or Council Staff and is dependent on the level of input required by the Drainage Committee of Council Staff. These inspections are in many cases carried out in-conjunction with the formulation of the yearly cleaning programme. Selwyn District Council require Drainage Committees and individual property owners to advise when maintenance is required outside of normal maintenance cycles, and such reporting is essential to augmenting the monitoring program.

Inspection and monitoring will be directed at identifying the following problems:

- Encroachment of weeds and other obstructions which may impair waterway performance
- Bank erosion that may also threaten the waterway performance, resulting in loss of pasture and sedimentation of the drains
- Accumulation of silt which may result in poor hydraulic performance
- Excessive bed degrading that may destabilise drain beds and banks.

Management Approach to Drain Cleaning

The efficient movement of water in the drain system requires an effective programme of control of aquatic and drain bank weeds and silt removal. Aquatic weeds and silt build up contribute to inefficient water use by restricting water movement in drains. However, stronger attention needs to be given to vegetation management, which includes balancing beneficial aspects of drain vegetation (i.e. minimise bank erosion, enhance amenity and provide habitat for fish /invertebrate and wildlife) with potential negative affects discussed above.

The purpose of the drain cleaning is to only remove the vegetation growing on the banks of the drain that restricts water flow and silt, sand and weed that has been deposited in the bottom of the drain and is reducing the waterway area or slowing water velocity. Drain cleaning is carried out using a combination of Mechanical cleaning, spraying and cutting.

Mechanical Drain Cleaning

The majority of drain cleaning occurs on an annual basis. The optimum time (economically) for cleaning is just prior to any high rainfall period as this reduces the time for weeds to re-establish. The limiting factors for drain cleaning to occur are:

- Not occurring during harvesting of crops
- Appropriate ground conditions i.e. not boggy that reduces the efficiency and has a detrimental effect on the soils.
- Number of complaints i.e. High St Leeston, with property owners requesting drain cleaned prior to Christmas due to aesthetics rather than a “drainage problem”.

To ensure mechanical drain cleaning operation is efficient and takes into consideration stakeholders the Council will:

- Use drain cleaning equipment and methods to minimise the effects of drain cleaning on fisheries and aquatic habitats.
- Use contractors that have staff with proven skills and experience or can demonstrate that they have the skills and suitable machinery for drain cleaning to ensure minimum adverse effect on waterways.
- Continue to develop and improve drain cleaning methods /approaches in consultation with stakeholders. It is recognised that this will take time and the Land Drain Subcommittees is committed to improving drain-cleaning activities.
- Document and refine protocols for mechanical cleaning.

Vegetation Management

Some vegetation growth (e.g., perennial grasses) is desirable on drain banks at and above the waterline to minimise establishment of land weeds and preventing drain bank wind and water erosion. The Council will:

- Investigate opportunities for enhancing drain planting
- Promote awareness of drain planting to private property owners
- Continue consultation with stakeholders on drain weed spraying activities
- Continue to develop the SDC Weed Spraying Contract for improving environmental performance of weed spraying activities
- Develop and incorporate “Best Practice” weed spraying protocols.

Stronger attention may need to be given to vegetation management, which includes balancing beneficial aspects of drain vegetation (i.e. minimise bank erosion, enhance amenity and provide habitat for fish /invertebrate and wildlife) with potential negative effects discussed above.

Willow Removal

Willow growth within the drainage system has been noted as a concern by Council Staff. Investigation into the extent and effect of the willow trees is required to ensure that any removal programme is effective and efficient. (E3)

Weed Cutting

Weed cutting only occurs in the LII Drainage scheme on the LII. Cleaning normally starts in late January and takes about 12 days. Cleaning is carried out using a floating weed cutting plant referred to as the Ellesmere Queen. Resource consents relating to the installation of a boom to retain weed in the LII were obtained in 1999.

Alternatives for Sedimentation and Vegetation Removal

The method for cleaning has been historically based focused on ensuring hydraulic efficiency. It is now recognised that drains may be important habitats, and have multiple uses and values. Many landowners recognise the values of managed waterways and want to do manage the environment responsibly.

Whatever the main interest, the long-term solution is preferably sustainable with respect to hydraulic performance, reduced costs, and multi-uses. Other reasons why changes in land drainage management are required include:

- Legislation, such as the RMA and Environment Canterbury NRRP
- Trade implications including the potential for imposition of non-tariff trade barriers by competing countries that are already operating under stricter environmental regulatory controls than New Zealand.

An investigation into sedimentation and vegetation management for the Drainage Systems is considered appropriate and necessary. (E1)

H. Resource Consent Conditions

This appendix contains summaries of the Resources Consent conditions relevant to weed removal of the L2 River, obtained from Environment Canterbury website.

RecordNo CRC000818

Consent Summary

Type Consent

Source Applic /New

Section

FileNo CO6C/16404

ClientName Selwyn District Council

To to erect, reconstruct and use a structure across the LII River, at or about map reference NZMS 260 M36:6565-2180, for the purpose of capturing cut weed.

Location Lii River, Lake Ellesmere

Events	22 Nov 1999	Consent Commenced
	19 Nov 2001	Consent Given Effect To
	22 Nov 2001	Lapse Date if not Given Effect To
	19 Nov 2034	Consent Expires

Subject to the following conditions:

- 1 The boom structure shall be erected, reconstructed and used in accordance with the details of the Selwyn District Council's facsimile dated 18 November 1999, submitted as part of this consent application.
- 2 The boom structure shall be under constant supervision while weed is accumulating, in accordance with the details provided in the consent application.
- 3 In the event that the boom structure or weed or other accumulated material causes water levels to rise to a height greater than 0.4 metres above the water level immediately downstream of the boom, or if the available freeboard is reduced by more than 0.2 metres further than 500 metres upstream of the boom, the consent holder shall immediately remove accumulated weed and other material.
- 4 The consent holder shall maintain a record of each occasion accumulated material is released from the boom for flood relief purposes.
- 5 Whilst the boom structure is in place, the consent holder shall do the following.(a) Maintain signs at the Wolfes Road Boat Ramp, Days Road and at a point 20 metres downstream of the boom structure. The signs shall warn river users of the presence of the boom structure and shall be capable of being read at a distance of at least five metres.(b) Attach a light to the boom structure during hours of darkness so that it can be clearly seen.
- 6 The boom structure shall not be used during the first two weeks of May or October in each year.
- 7 At least seven days prior to the use of the boom structure, the consent holder shall advise Te Runanga o Taumutu, Te Runanga o Ngai Tahu and Fish and Game New Zealand of their proposed weed control programme.

- 8 The Canterbury Regional Council may, on the last working day of November each year, serve notice of its intention to review the conditions of this consent for the purposes of: (a) dealing with any adverse effect on the environment which may arise from the exercise of this consent and which it is appropriate to deal with at a later stage; or (b) requiring the adoption of the best practicable option to remove or reduce any adverse effect on the environment; or (c) complying with the requirements of a relevant rule in an operative regional plan. (d) relocating the boom further downstream if significant adverse effects occur in the LII River or Lake Ellesmere/Te Waihora from the cut and release of aquatic weed.
- 9 Charges, set in accordance with section 36 of the Resource Management Act 1991, shall be paid to the Regional Council for the carrying out of its functions in relation to the administration, monitoring and supervision of resource consents and for the carrying out of its functions under section 35 of the Act.

RecordNo CRC000819**Consent Summary****Type** Consent**Source** Applic /New**Section****FileNo** CO6C/16404**ClientName** Selwyn District Council

To to discharge cut weed into Lake Ellesmere/Te Waihora at or about map reference NZMS 260 M36:6624-2001.

Location Lii River, Lake Ellesmere

Events	22 Nov 1999	Consent Commenced
	19 Nov 2001	Consent Given Effect To
	22 Nov 2001	Lapse Date if not Given Effect To
	19 Nov 2034	Consent Expires

Subject to the following conditions:

- 1 The discharge shall only comprise cut weed downstream of the boom structure authorised by consent CRC000818 and 10 percent slippage material as described in the consent application.
- 2 A record of complaints relating to the discharge of weed shall be maintained, and shall include:(a) location where the weed was encountered by the complainant;(b) date and time where the weed was encountered;(c) the most likely cause of the weed discharge; and(c) any corrective action undertaken by the consent holder to avoid, remedy mitigate adverse effect.This record shall be provided to the Canterbury Regional Council by 1 June each year, and otherwise on request.
- 3 All practicable measures shall be undertaken to minimise adverse effects on property, amenity values, wildlife, vegetation and ecological values.
- 4 The Canterbury Regional Council may, on the last working day of November each year, serve notice of its intention to review the conditions of this consent for the purposes of:(a) dealing with any adverse effect on the environment which may arise from the exercise of this consent and which it is appropriate to deal with at a later stage; or(b) requiring the adoption of the best practicable option to remove or reduce any adverse effect on the environment; or(c) complying with the requirements of a relevant rule in an operative regional plan.
- 5 Charges, set in accordance with section 36 of the Resource Management Act 1991, shall be paid to the Regional Council for the carrying out of its functions in relation to the administration, monitoring and supervision of resource consents and for the carrying out of its functions under section 35 of the Act.